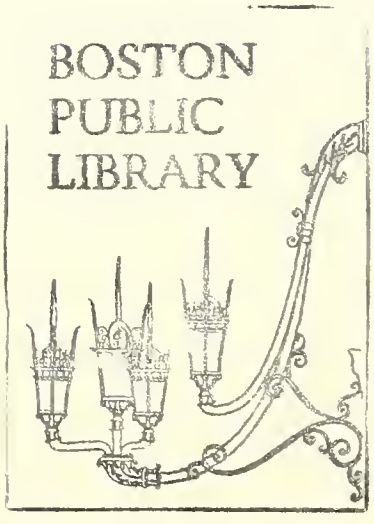
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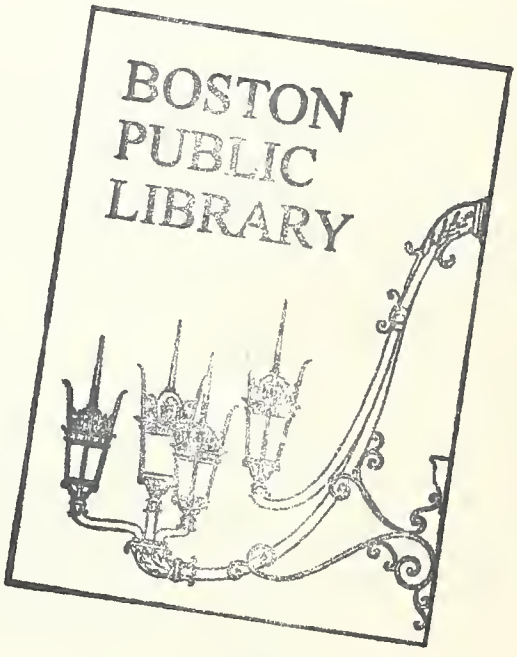
# REPORT TO THE METROPOLITAN DISTRICT COMMISSION ON COMBINED SEWER OVERFLOWS IN THE DORCHESTER BAY AREA

## VOLUME I - ENGINEERING REPORT

June 9, 1980




CAMP DRESSER & McKEE INC.
Environmental Engineers
Boston, Massachusetts

The Commonwealth of Massachusetts
Metropolitan District Commission
20 Somerset Street, Boston 02108

ENVIRONMENTAL
PLANNING OFFICE

TEL. 727-8880

GOVDOC

June 20, 1980

Mr. Wallace Orpin, Director of Engineering
Boston Redevelopment Authority
City Hall
Boston, Ma. 02201

Re: Dorchester Bay CSO Facilities Planning

Dear Mr. Orpin:

Enclosed is one copy of the draft facilities plan for control of Combined Sewer Overflows into Dorchester Bay (Volumes I & II).

Sincerely yours,

Francis J. Hopcroft
Director of Environmental Planning

Enclosure

FJH/ng

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Table of Contents


| | Page No. |
|---|---|
| CHAPTER I. SUMMARY OF CONCLUSIONS AND RECOMMENDATIONS | I-1 |
| Conclusions | I-1 |
| Recommendations | I-2 |
| Total Study Area | I-2 |
| South Boston Area | I-2 |
| Dorchester Area | I-3 |
| Miscellaneous Recommendations | I-3 |
| CHAPTER II. INTRODUCTION | II-1 |
| Background | II-1 |
| Regulatory Agency Requirements | II-1 |
| Project Area Relationships | II-3 |
| Facilities Planning Area | II-4 |
| Water Quality Issues and Objectives | II-5 |
| Purpose and Scope | II-5 |
| Project Approach | II-6 |
| Report Organization | II-6 |
| CHAPTER III. GENERAL DESIGN CRITERIA | III-1 |
| General | III-1 |
| Planning Period | III-1 |
| Land Use and Development | III-1 |
| Existing Land Use | III-2 |
| Future Land Uses | III-6 |
| Population and Water Consumption | III-7 |
| Population | III-7 |
| Water Consumption | III-8 |
| Existing Water Quality Standards | III-10 |
| Existing Water Quality | III-11 |
| Criteria for Analysis of Existing Combined Sewers and Facilities | III-13 |
| Data Sources | III-13 |
| Determination of Physcial Characteristics of Existing Sewage System | III-14 |
| Design Criteria of Existing Facilities | III-15 |
| Design Criteria of Recommended Facilities | III-16 |

CHAPTER IV. EXISTING FACILITIES IV-1

General IV-1
Combined Sewer Service Areas IV-1
Principal Interceptors, Combined Sewers, and Overflow Facilities IV-1

General IV-1
Combined Sewer System Operation IV-4
Principal Interceptors IV-4
Columbus Park Connection IV-8
Columbus Park Headworks IV-8
Calf Pasture - Moon Island Facilities IV-10
Regulators IV-13
Tide Gates IV-15
Combined Sewer Overflow (CSO) Outlets IV-15

Performance of Existing Facilities IV-17

Overall Performance IV-17
Maintenance - Related Problems IV-18
Direct Dry Weather Overflows IV-19
Sediment Buildup in Sewers IV-20

CHAPTER V. DRY WEATHER FLOWS V-1

General V-1
Field Inspection, Gaging and Sampling Program V-1

Description of Gaging Equipment V-2
Gaging Locations V-3
Gaging and Sampling Procedures V-8

Existing Dry Weather Flow Quantity V-9

General V-9
Dry Weather Flow Quantification Procedure V-9
Measured Dry Weather Flows V-14
Comparison of Calculated and Measured Dry Weather Flows V-18
Average Dry Weather Flow Balance V-18

Existing Dry Weather Flow Quality V-20
Dry Weather Discharges Through Combined Sewer Overflow Outlets V-22

CHAPTER VI. WET WEATHER FLOWS VI-1

Field Inspection, Gaging and Sampling Program V-1

Description of Equipment V-1
Gaging and Sampling Locations V-2
Gaging and Sampling Procedures V-2
Gaging and Sampling Events V-4
Wet Weather Flow Quality V-6

CHAPTER VII. COMBINED SEWER MODELING VII-1

Introduction VII-1
Selected Modeling Tools VII-1
The RUNOFF Model VII-3

Calibration VII-3
Use of Model VII-15

The SEMSTORM Model VII-15

Operation VII-15
Use of Model VII-17
Climatological Data VII-17
Runoff Characteristics VII-17
Quality Parameters VII-23
Output VII-23

The TRANSPORT Model VII-23

Operation VII-23
Application to Dorchester Bay Area VII-27
Verification VII-37

CHAPTER VIII. BOSTON HARBOR WATER QUALITY MODELING VIII-1

General VIII-1
Previous Work VIII-1

Steady State Model of Boston Harbor VIII-1
Hydrodynamic, Time Variable Water Quality Model VIII-2

Steady State Model VIII-3

General VIII-3
Advective Flows VIII-3
Dispersive Transport VIII-6
Calibration of Transport Characteristics VIII-6

Methodology for Analysis of Receiving Water Coliform Bacteria Concentrations VIII-7

Introduction to Statistical Method of Analysis VIII-7

Calibration of the Statistical Model VIII-8

Data Analysis VIII-8
Dry Weather Calibration VIII-9
Composite Calibration VIII-9

Table of Contents (Cont.)


| | Page No. |
|---|---|
| Hydrodynamic, Time-Variable Water Quality Models | VIII-10 |
| Model Geometry | VIII-10 |
| Application to Boston Harbor | VIII-12 |
| CHAPTER IX. ASSESSMENT OF EXISTING AND FUTURE NO-ACTION SITUATIONS | IX-1 |
| General | IX-1 |
| Quantity and Quality of Overflow | IX-1 |
| Basis of Load Determinations | IX-1 |
| Existing Situation | IX-6 |
| Future No-action Situation | IX-8 |
| Estimation of CSO Impact on Dorchester Bay | IX-8 |
| Assessment Criteria | IX-8 |
| Long-Term Assessment | IX-9 |
| Short-Term Assessment | IX-15 |
| Assessment of Future No-Action Situation | IX-24 |
| Summary | IX-24 |
| CHAPTER X. DEVELOPMENT AND SCREENING OF ALTERNATIVES | X-1 |
| General | X-1 |
| Study Area Alternatives | X-1 |
| Phase 1 Screening | X-1 |
| Phase 2 Screening | X-6 |
| Criteria for Phase 2 Screening | X-6 |
| Structural Alternatives | X-8 |
| Unit Processes | X-8 |
| General-Storage and Treatment | X-8 |
| Preliminary Storage-Treatment Analysis | X-9 |
| Preliminary Selection of Storage-Treatment Alternatives | X-18 |
| Screening of Storage-Treatment Alternatives | X-22 |
| Disinfection | X-31 |
| Combined System Separation | X-33 |
| Beach Enclosures | X-34 |

Best Management Practices X-35

General X-35
Screening of Effective Best Management Practices X-38

Alternatives Selected for Detailed Evaluation X-41

Structural Alternatives X-41
Best Management Practices X-42

Moon Island Joint Area Alternative X-42

General X-42
Description of Alternatives X-43
Estimated Costs of Alternatives X-45
Receiving Water Impact Determination X-46
Comparison with Study Area Alternatives X-47
Results of Comparison X-50
Conclusions X-50

Existing Deer Island Tunnel System Maximization Alternative X-53
General X-53
Pumping Capacity Increase X-53
Interceptor Capacity Increase X-55

CHAPTER XI. EVALUATION OF SELECTED ALTERNATIVES XI-1

General XI-1
Development of Selected Alternatives XI-1

General XI-1
Structural Alternatives - Storage/Containment XI-2

South Boston XI-4
Dorchester - Fox Point XI-11

Alternative D-A XI-11
Alternative D-B XI-15
Alternative D-C XI-16
Comparison of Fox Point Storage/Containment Alternatives XI-17
Dorchester - Commercial Point XI-17

Alternative D-E XI-20
Alternative D-F XI-21
Alternative D-G XI-22
Alternative D-H XI-23
Alternative D-I XI-24
Alternative D-J XI-24
Comparison of Commercial Point Storage/Containment Alternatives XI-25

Dorchester - Entire Area (Alternative D-L) XI-25

Structural Alternatives - Screening/Disinfection XI-30

South Boston XI-31
Dorchester - Fox Point and Commercial Point XI-33

Selection of Recommended Structural Alternatives XI-38

South Boston XI-38
Dorchester XI-38

Additional Recommendations XI-40

Hoyt Street Regulator Modifications XI-40
Pine Neck Creek Storm Drain Relocation XI-40

Non-Structural Alternatives XI-42

Dry Weather Overflow Abatement Program XI-42
Best Management Practices (BMP's) XI-44

Summary XI-45

CHAPTER XII. RECOMMENDED PLAN XII-1

General XII-1
Preliminary Design of Structural Recommendations XII-2

General XII-2
South Boston XII-2

Use of Unified Interceptor Model XII-3
Hydraulic Design Criteria XII-5
Siting XII-9
Consolidation Conduit XII-12

General Description and Design Criteria XII-12
Foundation Requirements and Construction Procedures XII-30
Estimated Costs XII-30

Storage/Containment Facility XII-30

Summary of Design Criteria XII-30
General Description XII-30
Superstructure XII-37
Foundation Requirements and Construction Procedures XII-38
Operation and Maintenance Requirements XII-38
Estimated Costs XII-39

Table of Contents (Cont.)


| | Page No. |
|---|---|
| CSO Outlet BOS-080 | XII-41 |
| Dorchester | XII-43 |
| Hoyt Street Regulator Modifications | XII-43 |
| Fox Point and Commercial Point Screening/Disinfection Facilities | XII-47 |
| Design Storm Selection | XII-47 |
| Hydraulic Design Criteria | XII-48 |
| Summary of Design Criteria | XII-50 |
| Siting and General Layout | XII-50 |
| Flow Diversion and Regulation | XII-54 |
| Screening | XII-55 |
| Disinfection | XII-60 |
| Heating, Ventilation and Odor Control | XII-62 |
| Superstructure | XII-62 |
| Foundation Requirements and Construction Procedures | XII-63 |
| Operation and Maintenance Requirements | XII-63 |
| Estimated Costs | XII-64 |
| Pine Neck Creek Storm Drain Relocation | XII-64 |
| General Description and Design Criteria | XII-64 |
| Foundation Requirements and Construction Procedures | XII-67 |
| Estimated Costs | XII-68 |
| Non-Structural Recommendations | XII-68 |
| Dry Weather Overflow Abatement Program | XII-68 |
| Best Management Practices | XII-69 |
| Miscellaneous Recommendations | XII-70 |
| Cleaning of CSO Conduits | XII-70 |
| Dredging | XII-71 |
| Landfill at CSO Outlet BOS-090 | XII-72 |
| Residual Waste Management | XII-72 |
| Impact of Recommended Plan on Dorchester Bay's Water Quality | XII-75 |
| General | XII-75 |
| Long Term Assessment | XII-75 |
| Short Term Assessment | XII-81 |
| Short Term Assessment - Southern Section of Dorchester Bay | XII-85 |
| Summary of Estimated Costs of Recommended Plan | XII-93 |

Table of Contents (Cont.)


| | Page No. |
|---|---|
| CHAPTER XIII. FINANCING AND IMPLEMENTATION | XIII-1 |
| General | XIII-1 |
| Construction Staging Program | XIII-1 |
| Financing Sources and Methods | XIII-1 |
| Eligibility and Financing Sources | XIII-3 |
| MDC Sewerage Division Cost Apportionment | XIII-5 |
| Expected Method of MDC Financing | XIII-6 |
| Municipal Sewer User Charges | XIII-6 |
| Expected Method of Municipal Financing | XIII-8 |
| Estimated Charges to Typical Residential Customers | XIII-9 |
| Present Charges | XIII-9 |
| Estimated Additional Charges for the CSO Project | XIII-10 |
| Estimated Charges for Study Area CSO Controls | XIII-10 |
| Legal and Institutional Issues | XIII-11 |
| Regulatory Agency Jurisdiction, Requirements and Permits | XIII-11 |
| Property Acquisition | XIII-13 |
| Legislation | XIII-14 |
| Recommended Implementation Plan | XIII-15 |

APPENDICES

Appendix A

Field Inspection and Flow Monitoring Results
Conduit (Link) and Junction (Node) Data for Transport Block

Appendix B

Report on the Unified Interceptor Modeling

Appendix C

Reports by Haley and Aldrich, Inc. on Foundation Requirements and Construction Procedures for Major Structural Recommendations

Appendix D

Bibliography

## List of Figures


| Figure | Title | Page No. |
|---|---|---|
| II-1 | Combined Sewer Overflow Project Planning Areas | II-2 |
| III-1 | Study Area Neighborhoods | III-4 |
| IV-1 | Schematic of Existing Sewerage System Showing Major Combined Sewers | IV-3 |
| IV-2 | Schematic Showing Combined Sewer System Operation | IV-5 |
| IV-3 | Schematic of Columbus Park Headworks | IV-9 |
| IV-4 | Calf Pasture - Moon Island Facilities | IV-11 |
| IV-5 | Types of Regulators Within Dorchester Bay Study Area | IV-14 |
| V-1 | Field Monitoring and Sampling Locations | V-5 |
| V-2 | Field Monitoring and Sampling Locations | V-6 |
| V-3 | Field Monitoring and Sampling Locations | V-7 |
| V-4 | Dry Weather Flow Quantification Procedure | V-10 |
| V-5 | Schematic of Estimated Average Dry Weather Flow Inputs to the Dorchester Bay Study Area Interceptor System | V-13 |
| VII-1 | Dorchester Bay Study Area | VII-2 |
| VII-2 | Computer Modeling of Study Area: Input, Simulation Functions, Computer Code and Output | VII-4 |
| VII-3 | Interrelation of Modeling Components | VII-5 |
| VII-4 | The Runoff Model's Mathematical Conceptualization of a Physical Drainage Area | VII-6 |
| VII-5 | Hyetograph for Storm of August 6, 1978 | VII-7 |
| VII-6 | Hyetograph for Storm of October 6, 1978 | VII-8 |
| VII-7 | Calibration Areas for Runoff Model | VII-9 |
| VII-8 | Simulated and Observed Hydrograph for the Storm of August 6, 1978 at K and Marine Streets, South Boston | VII-13 |
| VII-9 | Simulated and Observed Hydrograph for the storm of October 6, 1978 at Josephine Street, Dorchester | VII-14 |

List of Figures (Cont.)


| | | Page No. |
|---|---|---|
| VII-10 | Conceptualization of Urban Hydrology in SEMSTORM | VII-16 |
| VII-11 | SEMSTORM Sub-basins Used for Assessment of Existing Conditions | VII-18 |
| VII-12 | Aggragrated SEMSTORM Sub-Basins Used for Assessment of Future Conditions | VII-19 |
| VII-13 | Runoff Coefficient as a Function of Percent Imperviousness | VII-22 |
| VII-14 | Manner in Which SEMSTORM Results are Displayed | VII-25 |
| VII-15 | Representation of the TRANSPORT Model Link - Node Conceptualization | VII-26 |
| VII-16A | Runoff Sub-basins | VII-28 |
| VII-16B | Runoff Sub-basins | VII-29 |
| VII-16C | Runoff Sub-Basins | VII-30 |
| VII-17A | Schematic Representation of the South Boston Sewerage System for Use in the TRANSPORT Model | VII-33 |
| VII-17B | Schematic Representation of the Dorchester Sewerage System for Use in the Model TRANSPORT | VII-34 |
| VIII-1 | Steady-State Model Segmentation | VIII-4 |
| VIII-2 | Steady-State Model Flow Routing | VIII-5 |
| VIII-3 | Time-Variable Model Segmentation | VIII-11 |
| IX-1 | Map Showing SEMSTORM Areas and Harbor Elements | IX-3 |
| IX-2 | Tidal Currents in Dorchester Bay | IX-18 |
| IX-3 | Approximate Frequency of Wind Direction in Dorchester Bay | IX-19 |
| IX-4 | Probable Result of CSO Wastewater Plume Driven by Onshore Winds | IX-20 |
| IX-5 | Physical Dilution of CSO Wastewater Plume with Distance for Offshore Winds | IX-22 |
| IX-6 | Approximate Dispersion and Dilution of CSO Waste Water Plume for Onshore Wind Conditions | IX-23 |
| X-1 | Limits of Potential Storage - Treatment Alternatives for Subarea A-1 | X-13 |
| X-2 | Limits of Potential Storage - Treatment Alternatives for Subarea A-2 | X-14 |

List of Figures (Cont.)


| | | Page No. |
|---|---|---|
| X-3 | Limits of Potential Storage - Treatment Alternatives for Subarea A-3 | X-15 |
| X-4 | Overflow Days vs. Storage - Treatment Curves for Dorchester | X-19 |
| X-5 | Overflow Days vs. Storage - Treatment Curves for South Boston | X-20 |
| X-6 | Dorchester Screened Alternatives | X-26 |
| X-7 | Violation Days vs. Cost for Dorchester Area Screened Alternatives | X-27 |
| X-8 | South Boston Screened Alternatives | X-29 |
| X-9 | Violation Days vs. Cost for South Boston Screened Alternatives | X-30 |
| X-10 | Malibu Bay Beach Enclosure | X-36 |
| X-11 | Tenean Bay Beach Enclosure | X-37 |
| X-12 | Moon Island Joint Area Alternatives | X-44 |
| X-13 | Existing Headworks and Tunnel System | X-54 |
| XI-1 | Disaggregation of Study Area for Detailed Evaluation of Alternatives | XI-3 |
| XI-2 | South Boston Storage/Containment Alternatives | XI-7 |
| XI-3 | Profile of Proposed South Boston Consolidation Conduit | XI-9 |
| XI-4 | South Boston Storage/Containment Facility (Alt. SB-B) | XI-10 |
| XI-5 | Evaluation of Selected Alternatives for South Boston | XI-12 |
| XI-6 | Dorchester Storage/Containment Alternatives | XI-14 |
| XI-7 | Fox Point Selected Storage/Containment Alternative (D-C) | XI-18 |
| XI-8 | Evaluation of Selected Alternatives for Dorchester Area Tributary to BOS-088/089 (Fox Point) | XI-19 |
| XI-9 | Evaluation of Selected Alternatives for Dorchester Area Tributary to BOS-090 (Commercial Point) | XI-26 |
| XI-10 | Commercial Point Storage/Containment Alternative (D-F) | XI-27 |

## List of Figures (Cont'd)


| | | Page No. |
|---|---|---|
| XI-11 | Evaluation of Selected Alternatives for Entire Dorchester Area (Tributary to BOS-088/089 and BOS-090) | XI-29 |
| XI-12 | South Boston Screening/Disinfection Alternative (SB-C) | XI-32 |
| XI-13 | South Boston Detention/Chlorination Facility (Alt. SB-C) | XI-34 |
| XI-14 | Dorchester Screening/Disinfection Alternatives | XI-35 |
| XI-15 | Commercial Point Screening/Disinfection Facility (Alternative D-K) | XI-37 |
| XI-16 | Hoyt St. Regulator Modifications | XI-41 |
| XI-17 | Profile of Possible Route of Pine Neck Creek Storm Drain Relocation | XI-43 |
| XII-1 | Recommended Structural Facilities for South Boston | XII-4 |
| XII-2 | Site Plan - South Boston Storage/Containment Facility | XII-11 |
| XII-3 | South Boston Consolidation Conduit, Sta. 0+00 to Sta. 16+10 | XII-13 |
| XII-4 | South Boston Consolidation Conduit, Sta. 16+10 to Sta. 32+30 | XII-14 |
| XII-5 | South Boston Consolidation Conduit, Sta. 32+30 to Sta. 48+30 | XII-15 |
| XII-6 | South Boston Consolidation Conduit, Sta. 48+30 to Sta. 61+80 | XII-16 |
| XII-7 | South Boston Consolidation Conduit, Sta. 61+80 to Sta. 76+20 | XII-17 |
| XII-8 | South Boston Consolidation Conduit, Sta. 76+20 to Sta. 90+40 | XII-18 |
| XII-9 | Proposed Connection of Outfall BOS-085 to Consolidation Conduit | XII-19 |
| XII-10 | Proposed Connection of Outfall BOS-086 to Consolidation Conduit | XII-20 |
| XII-11 | Consolidation Conduit Junction Structure at Outlet BOS-081 | XII-21 |
| XII-12 | Consolidation Conduit Junction Structure at Outlets BOS-082 and BOS-083 | XII-22 |
| XII-13 | Consolidation Conduit Junction Structure at Outlet BOS-084 | XII-23 |
| XII-14 | Regulator Modification for BOS-085 Connection to Consolidation Conduit | XII-24 |
| XII-15 | Regulator Modification for BOS-086 Connection to Consolidation Conduit | XII-25 |

## List of Figures (Cont'd)


| | | Page No. |
|---|---|---|
| XII-16A, B | Proposed Junction Structure on Consolidation Conduit at BOS-087 | XII-27 |
| XII-17 | South Boston Storage/Containment Facility (Plan) | XII-32 |
| XII-18 | South Boston Storage/Containment Facility (Section) | XII-33 |
| XII-19 | South Boston Storage/Containment Facility (Details) | XII-34 |
| XII-20 | Recommended Structural Facilities for Dorchester | XII-44 |
| XII-21 | Recommended Pine Neck Creek Storm Drain Relocation | XII-45 |
| XII-22 | Hoyt Street Regulator Modifications | XII-46 |
| XII-23 | Site Plan - Commercial Point Screening/Disinfection Facility | XII-52 |
| XII-24 | Site Plan - Fox Point Screening/Disinfection Facility | XII-53 |
| XII-25 | Proposed Bascule Gate at Commercial Point and Fox Point Screening/Disinfection Facilities (Schematic, Plan and Sections) | XII-56 |
| XII-26 | Commercial Point and Fox Point Screening/Disinfection Facilities | XII-57 |
| XII-27 | Pine Neck Creek Storm Drain Relocation | XII-66 |
| XII-28 | Calculated Long-Term Total Coliform 50th % Results - Existing Situation and Recommended Plan | XII-79 |
| XII-29 | Calculated Long-Term Total Coliform 80th % Results - Existing Situation and Recommended Plan | XII-80 |
| XII-30 | Time-Variable Modeling Grids | XII-82 |
| XII-31 | Time Variable Water Quality Results - 1-Year, 6-Hour Storm Event | XII-86 |
| XII-32 | Time Variable Water Quality Results - 3-Month, 4-Hour Storm Event | XII-87 |
| XII-33 | Time Variable Water Quality Results - 1-Month, 4-Hour Storm Event | XII-88 |
| XII-34 | Water Body Segments - Southern Section of Dorchester Bay | XII-89 |
| XII-35 | Dilution Ratios of Malibu Bay Basin and BOS-089 CSO Outlet for 1-Month, 4-Hour Storm Event | XII-92 |

## List of Figures (Cont.)


| | | |
|---|---|---|
| A | Plan of Existing Sewerage System (Key Plan and Legend) | In Pocket |
| B | Plan of Existing Sewerage System | In Pocket |
| C | Plan of Existing Sewerage System | In Pocket |
| D | Plan of Existing Sewerage System | In Pocket |
| E | Plan of Existing Sewerage System | In Pocket |


In separate envelope

List of Tables


| Table | | Page No. |
|---|---|---|
| III-1 | Existing Land Use | III-5 |
| IV-1 | Information on Drainage Sub-Areas | IV-2 |
| IV-2 | Combined Sewer Overflow Outlets Within the Dorchester Bay Study Area | IV-16 |
| V-1 | Summary of the Dry Weather Flow Gaging and Sampling Program for the Dorchester Bay Study Area | V-4 |
| V-2 | Estimated Existing Average Daily Dry Weather Flows for the Dorchester Bay Study Area | V-12 |
| V-3 | Results of Dry Weather Flow Monitoring Program for the Dorchester Bay Study Area | V-15 |
| V-4 | Results of Dry Weather Flow Sampling Program for the Dorchester Bay Study Area | V-21 |
| V-5 | Summary of Known Direct Dry Weather Discharges to Dorchester Bay | V-23 |
| VI-1 | Summary of the Wet Weather Flow Monitoring and Sampling Program for the Dorchester Bay Study Area | VI-3 |
| VI-2 | Results of the Wet Weather Flow Sampling Program for the Dorchester Bay Study Area | VI-7 |
| VI-3 | Results of Laboratory Analyses of Wet Weather Samples of Volatile Organics and Pesticides | VI-3 |
| VII-1 | Runoff Input Parameters | VII-11 |
| VII-2 | Runoff Parameters for Calibration Areas | VII-12 |
| VII-3 | SEMSTORM Hydrologic Parameters | VII-20 |
| VII-4 | SEMSTORM Evaporation Rates | VII-21 |
| VII-5 | SEMSTORM Quality Parameters | VII-24 |
| VII-6A | Area Data for RUNOFF Block - South Boston | VII-31 |
| VII-6B | Area Data for RUNOFF Block - Dorchester | VII-32 |
| VII-7A | Conduit Data for RUNOFF Block - South Boston | VII-35 |
| VII-7B | Conduit Data for RUNOFF Block - Dorchester | VII-36 |

List of Tables (Cont.) Page No.


| | | |
|---|---|---|
| IX-1 | Estimated Existing System Conveyance Capacity and In-System Storage Volumes for Each SEMSTORM Sub-Area | IX-5 |
| IX-2 | SEMSTORM Input Data and Results of the Existing and Future No-action Situations | IX-7 |
| IX-3 | Harbor Model Total Coliform Calculated Results for Existing Situation | IX-11 |
| IX-4 | Disaggregation of Harbor Model Calculated Total Coliform Results for Existing Situation | IX-12 |
| IX-5 | Percent Contribution of Harbor Model Calculated Total Coliform Results for Existing Situation | IX-13 |
| IX-6 | Harbor Model Calculated Dissolved Oxygen Concentrations for Existing Situation | IX-14 |
| IX-7 | Harbor Model Calculated Mean Total Suspended Solids Concentrations for Existing Situation | IX-16 |
| X-1 | Study Area CSO Control Techniques Screened | X-2 |
| X-2 | Comparison of Possible CSO Abatement Alternatives with Objectives | X-7 |
| X-3 | Results of Initial SEMSTORM Runs for Determination of Storage-Treatment Boundary Conditions | X-12 |
| X-4 | Harbor Model Calculated Total Coliform Concentrations for Te-Se and Te-Smax Conditions | X-17 |
| X-5 | Harbor Model Calculated Total Coliform Concentrations for Preliminary Storage - Treatment Alternatives | X-23 |
| X-6 | Comparison of Screened Study Area Structural Alternatives | X-25 |
| X-7 | Characteristics of Principal Stormwater Disinfection Agents | X-32 |
| X-8 | Harbor Model Calculated Total Coliform Concentrations for Moon Island Joint Area Alternatives | X-48 |
| X-9 | Comparison of Moon Island Alternative 1 with Individual Study Area Alternatives | X-51 |
| X-10 | Comparison of Moon Island Alternative 3 with Individual Study Area Alternatives | X-52 |

List of Tables (Cont.)


| | | Page No. |
|---|---|---|
| XI-1 | Evaluation of Selected Alternatives | XI-5 |
| XI-2 | Summary of Selected Program | XI-46 |
| XII-1 | Summary of Design Criteria for Consolidation Conduit | XII-28 |
| XII-2 | Summary of Design Criteria for the South Boston Storage/Containment Facility | XII-31 |
| XII-3 | Summary of Estimated Costs for the South Boston Storage/Containment Facility | XII-40 |
| XII-4 | Results of SWMM for 1-Year, 6-Hour Storm for Fox Point and Commercial Point CSO Conduits | XII-49 |
| XII-5 | Summary of Design Criteria for Commercial Point and Fox Point Screening/Disinfection Facilities | XII-51 |
| XII-6 | Summary of Estaimated Costs for Commercial Point and Fox Point Screening/Disinfection Facilities | XII-65 |
| XII-7 | Comparison of Data for Existing Situation and Recommended Plan | XII-76 |
| XII-8 | Calculated Long Term Total Coliform Results for Existing Situation and Recommended Plan | XII-78 |
| XII-9 | Time Variable Water Quality Modeling Calculated Results - Peak Coliform Concentrations During Design Storm Events for the Existing Situation and Recommended Plan | XII-83 |
| XII-10 | Time Variable Water Quality Modeling Calculated Results - Relative Effect of the Recommended Plan on Peak Concentrations During Design Storm Events | XII-84 |
| XII-11 | Summary of Estimated Capital and Annual Operation and Maintenance Costs of the Recommended Plan | XII-94 |
| XIII-1 | Construction Staging Program | XIII-2 |
| XIII-2 | Summary of Estimated Costs and Funding Sources for Recommended Plan | XIII-4 |
| XIII-3 | Schedule of Ratios of Contribution for Apportionment of MDC Sewerage Division Debt Service and Operation and Maintenance Cost | XIII-7 |

## List of Tables (Cont.)


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| A-1 | Inventory of Regulators |
| A-2 | Inventory of Tide Gates |
| A-3 | Summary of the MDC Tide Gate Rehabilitation Program for the Dorchester Bay Study Area |

# CHAPTER I - SUMMARY OF CONCLUSIONS AND RECOMMENDATIONS

## Conclusions

Based on a review and analysis of available data pertaining to the study area, and the results of field inspection programs, the following conclusions can be made:

1. The 2894 acre highly urban study area has a current population of about 92,700. It is estimated that 47.9 percent of the study area's total acreage is served by combined sewers, 35.9 percent is served by separate storm and sanitary sewers, and 16.2 percent of the area is coastal and unsewered, thus runoff flows over land directly to Dorchester Bay. Approximately 86 percent of the separate and storm sewers discharge through combined sewer outfalls.

2. There are 11 CSO outlets within the Dorchester Bay study area, many of which discharge into waters adjacent to public beaches. Seven (No's. BOS-081 through 087) discharge into Old Harbor, where Carson Beach and L Street Beach are located. Outlet BOS-088 discharges into Malibu Bay, the site of Malibu Beach, and outlet BOS-090 is located in Tenean Bay, just north of Tenean Beach. Outlet BOS-080 discharges into the Reserved Channel which is used as a shipping channel. Boating and harvesting of the area's shellfish beds are also affected by discharges of combined sewer overflows through these outlets.

3. Overall, the performance of the Dorchester Bay area wastewater collection system is good. Based on field investigations during the summer and fall of 1978, no surcharge conditions exist in any of the major interceptors, and all currently appear to convey flow at adequate velocities. No major structural deficiencies were discovered in the regulators, tide gates, and sewer manholes that were inspected during the course of this study. However, several maintenance - related problems were discovered, generally consisting of blockages within the sewers and regulators due to excessive sediment buildup. These maintenance problems and a small number of direct, dry weather connections to overflow conduits that were also discovered, result in several dry weather flow discharges to Dorchester Bay.

4. Statistical water quality parameters calculated indicate that, on a long-term basis, Dorchester Bay currently does not meet SB standards in terms of total coliform concentrations and floatables. Quality is degraded severely enough to close a majority of shellfish beds located within the study area. The magnitude and direction of the wind (i.e., dispersion) and the tide level at the start of a storm (i.e., advection and dilution) have the greatest effect on the water quality of the swimming beaches following an overflow event.

5. Statistical harbor modeling results show that continuous dry weather overflows have the greatest effect on the degradation of Dorchester Bay's water quality. Upon removal of continuous dry weather overflows, the calculated long-term total coliform concentrations of Dorchester Bay will meet SB standards at all bathing areas, except Tenean Beach. Floatable standards will continue to be violated.

6. Calculated long-term total coliform concentration results indicate that if dry weather overflows are removed, about 48 percent of the remaining concentrations is attributed to CSO's, 42 percent from separate storm drainage and direct runoff, and about 10 percent from the Neponset River.

7. The major benefits derived from the abatement of combined sewer overflows are short-term with regards to water quality standards. Small, long-term incremental water quality benefits are realized with increasing control levels of CSO's. Abatement procedures will increase the use of the existing bathing beaches following overflow events.

8. An evaluation of CSO abatement alternatives concludes that for the South Boston area, the estimated number of annual overflow events (56) could cost-effectively be reduced to four. In the Dorchester area, modifications to the existing system would reduce the number of annual overflow events by about 50 percent. Because of the location of the Dorchester CSO outlets with respect to the bathing beaches, the remaining overflows should be screened and disinfected.

## Recommendations

The recommendations of this facilities plan are summarized as follows.

### Total Study Area

- Perform a Dry Weather Overflow Abatement program, consisting of the field inspection and testing of the sewerage system to locate and correct existing continuous dry weather sanitary flow connections to the combined and separate storm drainage systems.

- Implement continuous system best management practices consisting of regulator and tidegate maintenance, and catch basin and sewer cleaning.

### South Boston Area

- Construct a 9,040-foot long consolidation conduit to intercept existing CSO outlets. The conduit, constructed in Day Boulevard, and ranging in size from 36-in. to 78-in. diameter, would collect CSO's upstream of the outlet's tidegates, and transport them to a storage/containment facility.

- Construct a 3.0 mil. gal. capacity storage/containment facility adjacent to the MDC's Old Colony police station at the southern section of Carson Beach. The facility would screen and store CSO's during overflow events up to the 3-month, 4-hour storm, and pump back the CSO's to the Columbus Park headworks when hydraulic capacity becomes available.

Dorchester Area

- Construct two screening/disinfection facilities, one to serve the Commercial Point combined system, and the other for the Fox Point system. The facilities would screen CSO's up to the 1-year, 6-hour storm event, and would chlorinate the overflows during the 3-month bathing season.

- Relocate the outlet of the existing Pine Neck Creek separate storm drainage system to the Neponset River. This recommendation would eliminate the discharge of stormwater into the waters of Tenean Beach, and it consists of approximately 1,400 feet of 78-in. diameter pipe.

Miscellaneous Recommendations

- Dredge existing deposits which have accumulated at the study area's CSO outlets.

- Remove deposits which have settled in the CSO outfall conduits in the Dorchester area.

- At the Commercial Point outlet (BOS-090), remove a section of the landfill that is located downstream of the outlet and construct a trapozoidal, rip-rapped outlet channel.

The total capital cost of these recommendations (in June 1979 dollars) is estimated at about $36.7 million dollars, with an estimated annual operation and maintenance expense of about $281,000. About 89 percent of the capital cost will be eligible for funding through federal and state grants, while the remainder of the capital and all of the operation and maintenance expenses will remain a local responsibility.

# CHAPTER II - INTRODUCTION

## Background

Projects for the control of combined sewer overflow have been undertaken by the Metropolitan District Commission and its predecessor commissions since the early 1900's. These projects have included the construction of marginal conduits which intercept combined sewer discharges, the construction of combined sewer overflow (CSO) treatment facilities, and the improvement of community tidegates on CSO outlets.

In 1977, the Metropolitan District Commission (MDC) initiated its combined Sewer Overflow Project. This project is a comprehensive effort for CSO control in the cities of Boston, Cambridge, Chelsea and Somerville, and the Town of Brookline. These communities form the combined sewer service area of the Metropolitan Sanitary District, a district of 43 communities for which the MDC provides wastewater collection and treatment.

The combined sewers in these communities were constructed largely between 1860 and 1900. Since that time, several investigations by the local communities, state and federal agencies have identified combined sewer overflow as a major source of pollution in Boston Harbor and its tributary rivers. The MDC's activities in CSO control have been in response to these investigations.

In 1976, the MDC completed the Eastern Massachusetts Metropolitan Area (EMMA) Wastewater Engineering and Management Study, which prioritized CSO control needs in relationship to other wastewater collection and treatment needs, and made recommendations for CSO control planning. The approach to planning in the CSO Project was based on the recommendations of the EMMA Study. Four planning areas were delineated, and these areas are shown on Figure II-1. The planning areas include the Inner Harbor, the Charles River Basin, Dorchester Bay, and the Neponset River Estuary.

This report describes the planning for the Dorchester Bay area. The relationship between this planning area and the other three areas is discussed later in this chapter, following a review of the intent of this report and the regulatory requirements under which it has been prepared.

## Regulatory Agency Requirements

This report has been prepared in fulfillment of provisions dated August 11, 1976, of the NPDES (National Pollution Discharge Elimination System) permit for the MDC*. In accordance with those provisions, this report is intended to meet the facilities planning requirements of the Environmental Protection Agency (EPA) Region I and of the Massachusetts Division of Water Pollution Control. Following the acceptance of this facilities planning report, federal and state construction grant assistance is anticipated for the recommended CSO controls described in Chapter XII.

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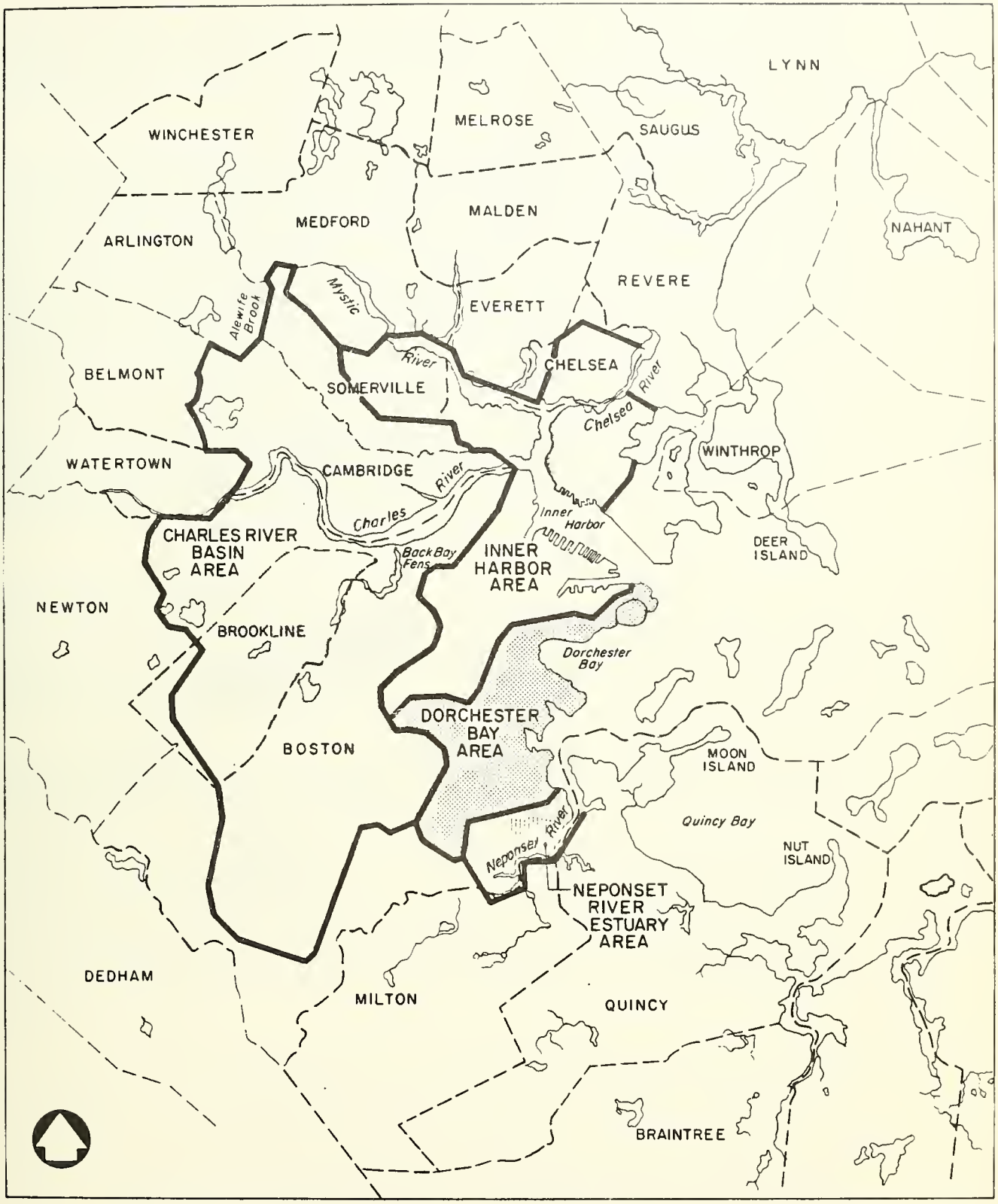
*NPDES number MA0102351, State number M-180.




COMMONWEALTH OF MASSACHUSETTS
METROPOLITAN DISTRICT COMMISSION
COMBINED SEWER OVERFLOW PROJECT
DORCHESTER BAY AREA
FACILITIES PLAN

FIG. II-1 COMBINED SEWER OVERFLOW PROJECT PLANNING AREAS.

The scope of work for this report (described later in this chapter) was developed early in 1978 to meet the EPA's facilities planning requirements at that time, as given in Title 40 of the Code, Federal Regulations Part 35, Subpart E.

With the passage of the Clean Water Act in 1977, the EPA amended its requirements for facilities planning on September 27, 1978. The EPA adopted new procedures for environmental assessments on December 15, 1979 in conformance with requirements of the Council on Environmental Quality. EPA also issued new regulations for public participation on February 16, 1979. During the planning study, changes were made in its scope to more closely comply with the new requirements for the environmental assessment and for public participation. The new requirements dealing with the environmental assessment are discussed in Volume 2, Chapter II, and those for public participation are described in Volume 2, Chapter VI.

The preparation of this facilities plan is consistent with the interim output of the Areawide Wastewater Management Plan (208), prepared by the Metropolitan Area Planning Council. The 208 Plan has recommended the control or elimination of combined sewer overflow, and has adopted the recommendations of the EMMA Study for CSO planning.

## Project Area Relationships

This section describes the general relationship among the four facilities planning studies which make up the MDC Combined Sewer Overflow Project. Following this section, the remainder of the report deals only with planning for the Dorchester Bay area.

Referring again to Figure II-1, the Combined Sewer Overflow Project area covers aabout 51 square miles bounded generally by the Mystic, Neponset, and Charles Rivers, and Boston Harbor. The combined sewers are tributary to the MDC Deer Island wastewater treatment plant.

The four planning areas were defined based on receiving water quality issues. The Dorchester Bay area is the primary water contact recreation area of Boston Harbor. The Charles River Basin, including the Back Bay Fens and Alewife Brook, is the most visible water resource in the Boston area. The Neponset River Estuary is the longest estuary entering Boston Harbor, and the Inner Harbor has the most commercial and shipping activity of the area.

The planning for the four areas was performed concurrently and by different consultants. The work was coordinated by the MDC with the assistance of a lead consultant. This approach to the planning was chosen so that the particular issues in each planning area could be studied in detail. The planning efforts were coordinated so that the CSO control objectives, the engineering and planning methods, and the recommended solutions are compatible. Interactions between planning areas were assessed through harbor water quality modeling (discussed in Chapter VII) and through coordination of the sewer modeling (discussed in Chapter VII). CSO control alternatives

which spanned two or more planning areas were also analyzed. Among these "joint area" alternatives were deep tunnel systems which are described in the Inner Harbor facilities plan, and use of existing tunnel and wastewater storage tanks at Boston's Moon Island, which is described in the Dorchester Bay facilities plan. Feasibility studies for the increased use of the Deer Island tunnel system to convey wet weather flow, and for the use of the Boston Main interceptor for CSO storage are also discussed in the Dorchester Bay facilities plan.

## Facilities Planning Area

As shown in Figure III-1 (see Chapter III of this volume), the Dorchester Bay CSO facilities study area includes major portions of the communities of Dorchester and South Boston. The total area consists of approximately 2900 acres. The planning area is bounded generally by Broadway and Dorchester Street to the north, Columbia Road and Washington Street to the west, and Ashmont Street to the south. The eastern border of the entire planning area is Dorchester Bay, a major recreational resource for the entire Boston community. Dorchester Bay, particularly its most northern section designated as Old Harbor, is the site of much water contact activity such as swimming, boating and fishing. There are several major beaches, parks and yacht clubs located within the study area for these purposes. In addition, extensive tidal flats along the coast contain productive shellfish beds where limited harvesting is permitted.

The primary land use within the Dorchester Bay study area is residential, most of which is densely populated. In coastal areas, the land use is more varied, including commercial, industrial, recreational, and open land as well as housing. Most of the significant industrial activity takes place within the Dorchester portion of the study area, along major thoroughfares such as the Southeast Expressway.

The study area is primarily served by a combined sewer system, with a few small areas having separate storm and sanitary sewers and also unsewered areas that contribute stormwater directly to receiving waters. During storm events, combined sewer overflows may discharge to Dorchester Bay at eleven overflow outlets within the study area. Historically, most of the wastewater flow from the entire Boston main drainage system passed through the Dorchester Bay study area as it was conveyed to the Calf Pasture pumping station, from which it was pumped to the detention tanks at Moon Island and eventually discharged to the outer harbor. Since the construction of the Deer Island wastewater treatment plant (1968), significant quantities of wastewater continue to enter the Dorchester Bay study area from the Inner Harbor and Neponset Estuary study areas, where flows combine with those from the major intercepting sewers in Dorchester and South Boston and are conveyed to the Columbus Park headworks (in South Boston) and subsequently routed to Deer Island. (The existing facilities are described in detail in Chapter IV of this volume.)

## Water Quality Issues and Objectives

The waters in Dorchester Bay are classified as SB by the Water Resources Commission, Massachusetts Division of Water Pollution Control. Waters assigned to this class are designated for the protection and propagation of fish, other aquatic life and wildlife; for primary and secondary contact recreation; and for shellfish harvesting with depuration. Dissolved oxygen concentration in SB water is established at a minimum of 6.0 mg/l, pH shall be in the range of 6.5-8.5 (not more than 0.2 units outside of the naturally occurring range), and total coliform bacteria are not to exceed a median value of 700 MPN per 100 ml with not more than 20 percent of the samples exceeding 1000 MPN per 100 ml during any monthly sampling period. Tables 1 and 2 in Appendix A of Volume II, and Chapter III, include a complete description of all water classification and the water quality standards associated with each.

The designated uses associated with Class SB are compatible with the present uses of Dorchester Bay previously mentioned. Water quality standards for this class must be achieved for both primary contact recreation (bathing) and shellfishing to take place in the Bay.

However, it is apparent from the past water quality surveys and sampling programs conducted for this project that Dorchester Bay waters do not consistently meet SB quality standards, especially along shoreline areas. Coliform concentrations are often exceeded in the vicinity of the bathing beaches immediately following significant rainfall events. Discharges of oil, grease and floating material, along with odors, frequently occur near recreational areas and yacht clubs, creating an aesthetic nuisance. Contamination of shellfish beds has severely limited the number of beds which remain open for harvesting of shellfish for human consumption.

Current water quality standards serve as the basis for determining limits to the quantity and quality of combined sewer overflows and other point or nonpoint pollutant discharges to the bay. Subsequent sections of this report will further define the present impacts of CSO's on Dorchester Bay quality and the degree of improvement gained through alternative abatement measures.

## Purpose and Scope

The purpose of this facilities plan is to define the extent of the CSO problems, including the impact of CSO's on water quality of Dorchester Bay, and to determine the most appropriate and cost-effective means of CSO abatement. Other sources of pollution to the Bay, such as stormwater runoff and continuous dry weather sanitary flow discharges, are also investigated for their effects on water quality relative to CSO's. Measures for abatement of dry weather flows and major stormwater discharges are included, as they are pertinent to the overall objectives of the planning effort.

## Project Approach

The initial stages of the project were devoted to the collection of existing data and generation of background information to be used throughout the study. This involved a review of past reports and existing maps and plans of the study area and its sewerage facilities. Maps were updated, when necessary, based on field investigations. Data on land use, drainage basins and physical characteristics of the sewerage system were assembled for input to the computer simulation models.

Field programs involving flow monitoring and sampling of sanitary, storm and combined sewers were performed in order to generate information on the quantity and quality of dry and wet weather flows in the Dorchester Bay study area. This data was also necessary to verify the computer models, which simulated the response of the system to storm events and provided overflow pollutant loads to a quality model fo the Boston Harbor receiving waters.

Having assessed the theoretical loadings from the study area and their impact on the water quality fo the Harbor, specific abatement methods were conceptualized and reviewed. Methods which survived preliminary screening were then subjected to a detailed analysis in terms of their implementability, cost-effectiveness, and load abatement implications. The computer simulation models aided in both the preliminary and detailed screening processes. The abatement methods which were determined to be the most cost-effective in terms of satisfying the water quality objectives were selected as part of the recommended plan. The recommended plan is designed to an extent suitable for input to Step 2 (detailed design) of the facilities planning process, and an indication of the costs and financing methods is offered.

## Report Organization

The following report is divided into four basic sections:

- Review and definition of the existing conditions (Chapters III-VI)
- Activities and results of computer simulation (Chapters VII-IX)
- Screening and evaluation of CSO abatement concepts (Chapters X and XI)
- Development and implementation of the recommended plan (Chapters XII and XIII)

# CHAPTER III - GENERAL DESIGN CRITERIA

## General

The analyses and recommendations that comprise a facilities planning study must be built on the basis of general design criteria--the standards on which planning decisions are based. The duration of the planning period, the existing and anticipated future population, water consumption and land uses within the study area, the uses and assimilative capacity of receiving waters, and the condition and design criteria of the existing wastewater system all must be defined and agreed to in order to develop informed and effective recommendations for the future. This chapter presents a general overview of the design criteria that form the basis of this facilities plan. More detailed discussions of the information presented here are contained in later chapters of Volume I and in Volume II-Environmental Assessment Report.

## Planning Period

The planning period for this study extends from 1985 to 2005. This is in accord wtih EPA Interim Cost-Effectiveness Analysis Guidelines,* which define the planning period as 20 years from the initial operation of the waste treatment system, and NPDES permits for the Metropolitan District Commission (MDC), which require that Combined Sewer Overflow (CSO) facilities be operational between 1983 and 1985.

The planning period defines the duration of time for which the waste treatment system is evaluated for cost effectiveness and the period for which population and wastewater flow projections are developed. Service life, useful life, and construction staging for CSO control facilities will be addressed later.

## Land Use and Development

Land use is directly related to the quantity of runoff expected to flow to receiving waters during wet weather events. The type and quantity of pollutants contained in runoff varies considerably from one land use category to another, and a thorough knowledge of existing and potential future land uses is an important element of the facilities planning study.

Depending on population densities, areas used for residential purposes may contribute high coliform concentrations to combined sewer overflows due to the relatively high volume of domestic sanitary sewage discharged

*Revised Appendix A to 40 CFR Part 35, Subpart E, in 43 CFR, dated September 27, 1978, and effective October 1, 1978.

into the combined sewer system. Industrial areas may pose additional problems including the discharge of heavy metals, volatile organics, polychlorinated biphenyls (PCBs), and other toxic substances that are found in many manufacturing processes and their waste streams. Industries may also add to the total volume of combined sewer overflow if they contribute a major portion of the dry weather flow.

Parks and other unpaved open areas serve to reduce the amount of runoff that drains to either a combined sewer or storm drain because much of the rainwater that falls on grass-covered areas is able to infiltrate into the ground. However, open areas or parks that are not properly graded, and/or do not have sufficient grass cover, can increase the grit and suspended solids content of the runoff as a result of soil erosion. This can lead to sediment deposits within conduits and a high solids content in CSOs and their receiving water.

Areas designated for institutional land use can increase the pollutional content of CSOs in several ways. Institutional areas containing schools, churches, hospitals, or municipal buildings have a potential for discharging large slugs of sewage and associated pollutants to the collection system. If these discharges occur during a rainfall event, the sanitary portion of the total flow in the combined sewer can increase, increasing the pollutant concentrations of CSOs, where they occur. Hospitals have the additional potential of producing waste streams containing high concentrations of pathogenic organisms that may become a health problem when discharged in the vicinity of bathing or shellfish areas.

Runoff from highways, major roadways and railroads must also be addressed. Oil, grease and other automotive pollutants are often present on road surfaces in sufficient quantities to contaminate stormwater runoff that eventually discharges to receiving waters. Hydrocarbons from engine exhaust systems plus sanding and salting operations during the winter season can also add contaminants to roadway runoff.

In summary, it is apparent that existing and future land uses in the study area have a direct bearing on the types and quantities of pollutants that may be discharged to the area's receiving waters (Dorchester Bay and Old Harbor) through combined sewer overflows. In order to locate and quantify sources of contaminants, land use must be examined in each subdrainage area and potential major pollutant sources identified.

Existing Land Use. Existing land use has been divided into seven categories that are defined as follows:

1. Commercial - stores, business establishments and shopping areas of any type;

2. Residential - homes and housing areas;

3. Industrial - areas occupied by manufacturing and processing uses, both light and heavy;

4. Institutional - schools, churches, hosptials, and comparable facilities;

5. Parks and Cemeteries - all parkland (i.e., Columbus Park, etc.) and cemeteries;

6. Major Transportation - major vehicular and railroad right-of-ways (i.e., Southeast Expressway, MBTA rapid transit system);

7. Open and Public Lands - other land use categories not included above.

As shown on Figure III-1, the study area is currently subdivided into two main neighborhoods: Dorchester and South Boston. Listed in Table III-1 is the estimated acreage of the seven land use categories within the neighborhoods and their acreage as a percentage of the total study area's acreage.

Land use within the study area is predominantly residential, primarily consisting of high density, multi-family dwellings. The Dorchester portion of the study area (tributary areas 087, 088 and 090 on Figures B, C and D, appended) contains relatively small areas of commerical and industrial uses as well as several public parks and playgrounds. Several public schools are located within the area, and the University of Massachusetts located on Columbia Point, comprises the majority of the institutional land use. Major transportation arteries include the Southeast Expressway, generally located parallel to the Dorchester shoreline, that provides vehicular access to and from downtown Boston from south shore communities, and Morrissey Boulevard, which extends between Neponset Circle and Columbia Circle. The Ashmont and Quincy branches of the MBTA's Red Line also pass through the Dorchester area, and access is provided to area residents at the Fields Corner, Savin Hill and Columbia stations.

The Dorchester area has two public beaches; (1) Malibu Beach, located between Savin Hill and Commerical Point in tributary area 089, and (2) Tenean Beach, located south of Commercial Point at the southern extremity of the study area in tributary area 090. The three yacht clubs in the Dorchester area are located at the three combined sewer overflow outlets serving this area. Referring to Figure C, appended, yacht clubs include the Savin Hill Yacht Club located at Fox Point (Outlet No. Bos-089), the Dorchester Yacht Club located in Malibu Bay (Outlet No. BOS-088), and the Old Colony Yacht Club, located in Tenean Bay south of Commercial Point (Outlet No. BOS-090). The effect of combined sewer overflow discharges on activities at the yacht clubs will be addressed later in this report.

The South Boston portion of the study area (tributary areas 080 through 086 on Figure B, appended) is comprised primarily of high density, multi-family dwellings. Major public parks in the area include Marine Park, located east of Farragut Road, and Columbus Park, situated between Columbia Road and William J. Day Boulevard. These latter two arteries are the major roadways serving the South Boston area and provide access to the area's beaches and yacht clubs. Both generally follow the shore-

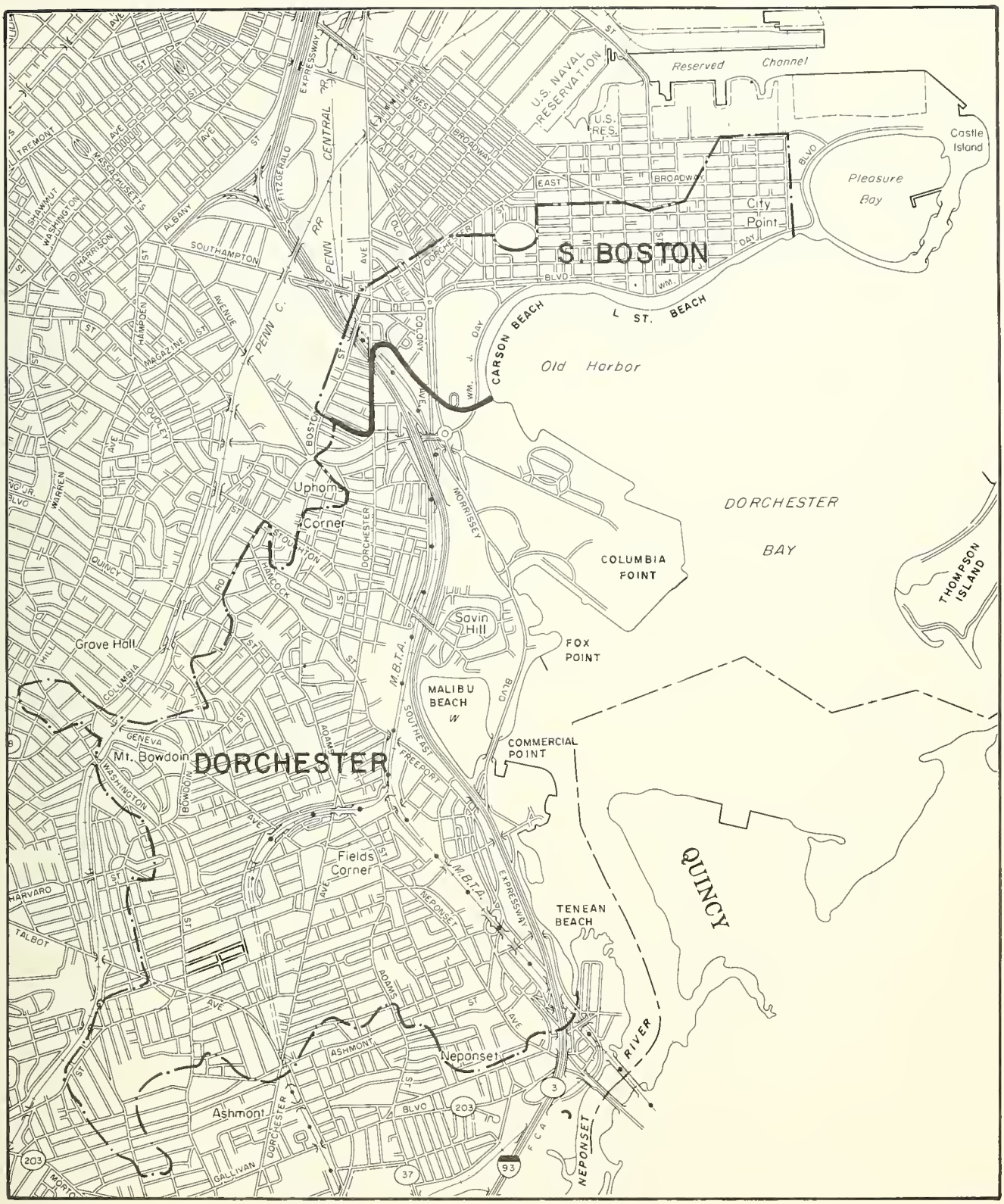



FIG. III-1 STUDY AREA NEIGHBORHOODS

COMMONWEALTH OF MASSACHUSETTS
METROPOLITAN DISTRICT COMMISSION
COMBINED SEWER OVERFLOW PROJECT
DORCHESTER BAY AREA FACILITIES PLAN

TABLE III-1. EXISTING LAND USE

| Land Use Category | South Boston Subarea | | Dorchester Subarea | | Total Study Area | |
|---|---|---|---|---|---|---|
| | Estimated* Area, acres | Percent of Total Study Area | Estimated* Area, acres | Percent of Total Study Area | Estimated* Area, acres | Percent of Total Study Area |
| Commercial | 90 | 3.1 | 246 | 8.5 | 336 | 11.6 |
| Residential | 274 | 9.5 | 1,439 | 49.7 | 1,713 | 59.2 |
| Industrial | - | - | 93 | 3.2 | 93 | 3.2 |
| Institutional | 10 | 0.3 | 116 | 4.0 | 126 | 4.3 |
| Parks and Cemeteries | 68 | 2.3 | 84 | 2.9 | 152 | 5.2 |
| Major Transportation | 6 | 0.2 | 193 | 6.7 | 199 | 6.9 |
| Open and Park Lands | 37 | 1.3 | 238 | 8.3 | 275 | 9.6 |
| Total | 485 | 16.7 | 2,409 | 83.3 | 2,894 | 100.0 |

*Planimetered from M.A.P.C. Land Use and Vegetative Cover Map (1971).

line and extend from Columbia Circle to Farragut Road with William J. Day Boulevard continuing to Castle Island. As shown on Figure B, appended, two of the area's three public beaches are within close proximity to combined sewer overflow outlets. Outlets BOS-084 through BOS-087 discharge into waters off of Carson Beach, while outlets BOS-082 and 083 discharge overflows near the L Street Beach. The remaining public beach is Pleasure Bay, whose waters are almost totally encircled by land. The water level of the bay is controlled by two weirs located on the causeway between Castle Island and Head Island. There are no combined sewer outlets located in Pleasure Bay. There are four yacht clubs in the South Boston area, all located between O Street extended and P Street extended as follows: (1) the South Boston Yacht Club, (2) the Columbia Yacht Club, (3) the Puritan Canoe Club, and (4) the Boston Harbor Yacht Club.

Future Land Uses. In general, it appears that only minor development and land use changes will take place in the study area during the planning period. Because of the nature of existing land uses, no large open areas where industry or large commercial establishments could develop are available. Industrial or commercial growth potential is limited to a few small areas zoned for this type of development (i.e., the proposed Alsen Mapes Industrial Park in Dorchester near Park Street). According to the Boston Redevelopment Authority (BRA), local citizen opposition to high-rise residential development is likely to negate higher density residential land uses as well. Unless complete urban renewal of large tracts of existing residential land areas is undertaken, which at this time seems highly unlikely, the study area's overall existing land uses are expected to remain relatively stable during the planning period.

On a local level, the only exception currently contemplated is the revitalization of the Columbia Point housing project. The project was opened in 1952 as a low income housing development, consisting of approximately 1,500 units. Crime, isolation and neglect have caused a migration of tenants, dropping the population of the development by 70 percent in the last decade. As currently conceived, the proposed revitalization project would, when coupled with the recent construction of the University of Massachusetts (Boston campus) and the J.F. Kennedy Memorial Library, make Columbia Point the focal point of residential development in the area. The proposed mixed-income development project would consists of the renovation or demolition of existing housing units and the construction of new, low-rise units. Additional redevelopment includes renovations to the existing Bayside Shopping Mall and improvements to the existing roadway system, waterfront transportation, and recreational facilities. A more detailed discussion of the revitalization program follows later.

## Population and Water Consumption

An analysis of the existing and estimated future population and water consumption of the study area is necessary to estimate the area's dry weather wastewater flow rates. Per capita water consumption estimates, obtained from metered water consumption data, multiplied by estimated population, results in an estimate of total average daily water consumption on a sub-area or study area basis. This information is used to estimate average daily domestic wastewater flows, and when added to estimates of industrial and institutional flows and infiltration, results in the estimated total average dry weather flow. This latter information is essential when estimating the quantity and quality of combined sewer overflows. Although the quantity of dry weather flow is relatively minor when compared to the large volumes of stormwater in combined sewer overflows, its high contaminant level has impacts on the quality of the overflows. A detailed discussion on dry weather flows including the quantifications procedure used for this facilities plan, is presented in Chapter V.

Population. Estimates of existing population for the Dorchester Bay study area were developed utilizing 1975 community population, information compiled by the Central Transportation Planning Staff (CTPS).[1] This data was disaggregated into traffic zones which, in most cases correspond to 1970 federal census tracts. Accordingly, the 1970 and 1975 study area populations were estimated by summing the population of each census tract (or traffic zone) located within the area's boundary. The disaggregations of population between the South Boston and Dorchester portions of the study area is summarized below:

| | South Boston | Dorchester | Total Study Area |
|---|---|---|---|
| Area, acres[2] | 388 | 2,039 | 2,427 |
| 1970 Estimated Population | 23,396 | 77,007 | 100,403 |
| 1970 Population Density, persons/acre | 60.3 | 37.8 | 41.4 |
| 1975 Estimated Population | 22,826 | 69,802 | 92,628 |
| 1975 Population Density, persons/acre | 58.8 | 34.2 | 38.2 |
| % Decrease-1970 to 1975 | 2.4 | 9.4 | 7.7 |

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[1] The CTPS is an interagency group created by the Executive Office of Transportation and Construction (EOTC), the Metropolitan Area Planning Council (MAPC), the Massachusetts Department of Public Works (DPW), the Massachusetts Bay Transportation Authority (MBTA), the Advisory Board to the MBTA, and the Massachusetts Port Authority. These six agencies have been designated collectively as the Metropolitan Planning Organization (MPO) for transportation purposes in the Boston region. MAPC administers the CTPS.

[2] Based on a summation of traffic zone areas located within the study area.

Examination of historical federal census data for the Dorchester Bay study area indicates that this decreasing population trend began during the 1950-1960 period. Regarding the study area's projected future populations, no projections were done specifically for Dorchester or South Boston by the CTPS. CTPS projections for the City of Boston* indicate a declining growth rate as follows:

| Year | Estimated Population (by CTPS)* |
|---|---|
| 1975 | 626,988 |
| 1980 | 640,876 |
| 1985 | 643,845 |
| 1990 | 646,814 |
| 1995 | 650,676 |
| 2000 | 654,537 |

Based on this expected decrease in population for the City of Boston as a whole, and the fact that there is a very limited amount of land available for new development in the study area, it is expected that the population in the Dorchester Bay study area will generally follow the CTPS projections for the City of Boston. However, in order to allow for any deviations from this expected future growth pattern, the 1975 estimated population at 92,628 as determined by CTPS has been adopted as the design year (2005) population for this facilities plan.

Water Consumption. Based on 1977 metered water consumption information obtained from the Boston Water and Sewer Commission it is estimated that the average per capita water consumption for the Dorchester Bay study area is about 72 gallons per day. Using the estimated 1975 study area population of 92,628 as discussed in the previous section, the average daily domestic water consumption for the study area is estimated to be about 6.67 mgd. Major industrial and institutional users were also identified, and their 1977 metered water use estimated. For the purposes of this study, any user with a 1977 water consumption volume greater than 100,000 cubic feet (748,000 gallons) was defined as a major user. These include the University of Massachusetts, Boston College High School, the Boston Globe, the First National Bank of Boston office building, and several launderies, car washes, small industries and commercial establishments. The 1977 average daily water usage for the areas's major industrial and institutional users was about 0.61 mgd. Accordingly, the 1977 total average daily water use in the study area is estimated to be about 7.28 mgd. This total has been increased by 20 percent to account for meter inaccuracies and unmetered usage (unaccounted-for water). The reasoning for this increase is discussed later in detail in Chapter V.

No significant increase in the study area's water consumption is expected in the future, based on the premise that no population increase will occur, and the amount of buildable land available for new industrial or commercial development is minimum. Per capita water consumption is also expected to remain at estimated existing levels (i.e., 72 gal/day/cap) due to the current emphasis on water conservation and the likely saturation of water consuming devices (i.e., garbage disposals, dishwashers) in area residences. Accordingly, the total estimated average daily water

*Projections made in October, 1979.

consumption of 7.28 mgd was adopted as representing existing as well as future (design year) conditions. Chapter V presents a detailed discussion of the use of water consumption data in estimating dry weather wastewater flows.

Existing Water Quality Standards

Dorchester Bay serves as a very important recreational resource for Boston area residents, especially those from South Boston and Dorchester. Since its entire shoreline is contained within the latter two communities, the Bay is easily accessible and often the focal point of local activities. Swimming, boating and fishing are some of the activities offered at numerous public beaches, yacht clubs and piers along the Bay. Shellfish harvesting is permitted in certain areas as well.

Water quality standards relating to these uses must serve as the basis for determining limits to the quantity and quality of combined sewer overflows and other point or non-point discharges to Dorchester Bay waters. The impacts that deterioration of the water quality of the Bay has had and will continue to have on the life, health, and aesthetic appearance of these neighborhoods must be considered in the evaluation of CSO alternatives.

The waters in Dorchester Bay are classified as SB by the Water Resources Commission, Masschusetts Division of Water Pollution Control. Waters assigned to this class are designated for the uses of protection and propagation of fish, other aquatic life and wildlife; for primary and secondary contact recreation and for shellfish harvesting with depuration.* Dissolved oxygen concentration in SB waters is established at a minimum of 6.0 mg/1, pH shall be in the range of 6.5 - 8.5 (and not more than 0.2 units outside of the naturally occurring range), and total coliform bacteria are not to exceed a median value of 700 MPN per 100 ml with not more than 20 percent of the samples exceeding 1,000 MPN per 100 ml during any monthly sampling period.**

Additional standards regarding shellfish harvesting also apply to the Dorchester Bay area. U.S. Public Health Service standards dictate that total coliform counts in water of 0-70 MPN per 100 ml indicate "clean" shellfish beds; 71-700 MPN per ml, "moderately contaminated" beds; and over 700 MPN per 100 ml, "grossly contaminated" beds. Grossly contaminated beds are closed to the harvest of shellfish for human consumption, but shellfish in moderately contaminated beds may be harvested with depuration.

*Depuration is defined as the reduction of fecal coliform bacteria in shellfish to levels suitable for marketing. This will be discussed later in this section.

**For a complete description of all water classifications and the water quality standards associated with each, see Vol. II Chapter III of this report, Appendix A, Tables 1 and 2.

The depuration process serves as another means of shellfish quality control. Clams from moderately contaminated beds may be brought by commercial fishermen to the Division of Marine Fisheries' Newburyport Depuration Center. Upon arrival, fecal coliform counts in the clams are checked, the maximum level permitted being 1,600 per 100 g. (gross weight) of clam. If this initial count is exceeded, the bed is closed for further harvesting. After the clams are flushed with fresh sea water for 48 hours, fecal coliform counts are again taken and the counts must be less than 230 per 100 grams of clam sample (gross weight) in order for the clams to be marketable.

While the above criteria represent state requirements for Class SB waters and federal requirements for shellfish, there are other water quality parameters that are perhaps equally as critical in evaluating appropriate use of Dorchester Bay. Biochemical Oxygen Demand (BOD) is an important indicator of pollution in a receiving water. Nutrient concentrations, particularly nitrogen and phosphorus, indicate the extent of eutrophiction potential. Heavy metals and other toxic chemicals (such as DDT, PCBs, etc.) can have a serious effect on the food chain. Fecal coliform levels are important because they are a more likely indicator of the presence of pathogens than are total coliform concentrations and because, as noted above, they are used in evaluating the quality of clams.

To our knowledge, the Food and Drug Administration (FDA) and the EPA have set few restrictions or limits on human ingestion of seafood contaminated by heavy metals, although they do provide information on human body symptoms to metals toxication. (Vol. II, Chapter III, Table III-6). There is also published data on concentrations of heavy metals lethal to marine organisms (Vol. II, Chapter III, Table III-8) and on EPA seawater quality criteria for heavy metals (Vol. II, Chapter III, Table III-6). In addition, concentrations of mercury greater than 1.0 ppm in the edible portion of seafood will cause the FDA to take legal action to remove it from the market. The FDA will also take action to remove seafood containing Dieldrin ( .3 ppm) and/or DDT or DDE ( 5 ppm) from the market (Vol. II, Chapter III, Table III-10). Where no information on tolerance levels exists, the FDA can, at their discretion, take legal action against a product at the minimum detectable level.

The designated uses associated with the SB classificationa are compatible with the present uses of Dorchester Bay mentioned earlier. As a minimum, water quality standards for this class must be achieved for both primary contact recreation (i.e., bathing) and shellfishing with depuration to take place within the Bay.

Existing Water Quality. Based on information obtained from past water quality surveys and sampling programs conducted for this project, it is apparent that, at the present time, Dorchester Bay waters do not consistently meet SB water quality standards, especially along shoreline areas immediately following signficant rainfall events. A detailed analysis of the available data on Dorchester Bay water quality can be found in Vol. II, Chapter III of this report.

Coliform bacteria appears to be the most serious contaminant affecting the quality of the Bay waters. Although the data reported varies widely with sampling location and information source, in general it appears that coliform concentrations tend to increase with rainfall and with proximity to the Neposnet River (near the southern boundary of the study area). During dry weather periods, the water quality in the vicinity of the South Boston beaches is generally good. Following a wet weather event, however, the coliform concentrations usually exceed the limitations of SB water quality. At Fox Point and at Malibu and Tenenan beaches in Dorchester, the coliform concentrations in coastal waters were found to periodically exceed the standards during both wet and dry weather periods. The lowest coliform concentrations were found in Pleasure Bay, where almost all reported coliform levels meet SB water quality standards.

There are approximately 300 acres of tidal flats in the Dorchester Bay area which are considered to be productive for soft-shell clams. Due to high coliform counts in the Bay waters, most of the shellfish beds (approximately 240 acres) cannot be harvested. All of the tidal flat areas, with the exception of Pleasure Bay, are currently classified as "grossly" contaminated and closed to the harvest of shellfish for human consumption. At Pleasure Bay, which is considered "moderately" contaminated, clams may be harvested, but only with depuration.

Dissolved oxygen concentrations were found to be substandard (less than 6.0 mg/1) at most of the coastal sampling stations. Offshore samples, however, showed higher DO values. All available data indicated that pH values in Dorchester Bay waters were within SB limits.

Data on other water quality parameters in Dorchester Bay appeared to be typical for an enriched coastal environment. The Bay was found to contain excessive phosphorus concentrations when compared to normal nitrogen/phosphorus ratios, (nitrogen and phosphorous are normally utilized by aquatic life in an average ratio of 5 parts nitrogen to 1 part phosphorus). Biochemical oxygen demand (BOD) varied considerably among sampling stations. The BOD of water column samples taken in the vicinity of the Old Harbor Beaches ranged from 1.0 to 57 mg/1, with an average of 6.9 mg/1. Samples taken from study area waters south of Old Harbor averaged 2.4 mg/1.

There appears to be limited data on heavy metals concentrations throughout Dorchester Bay. However, the limited data that are available indicate that water column and surface heavy metals concentrations are well below limits reported to be lethal (through laboratory experimentation) to certain marine organisms, including clams (Vol. II, Chapter III, Tables III-3, 4, 6 and 7). However, some average heavy metals concentrations in the water column, as well as some reported by the Inner Harbor Study Area consultant's surf sampling program, exceed EPA criteria for seawater, and typical mercury values exceed those reported by the New Jersey Department of Environmental Protection to be toxic to marine life (see Vol. II, Chapter III, Table III-7).

Accordingly, because heavy metals concentrations are at least partially in excess of published agency criteria, they should be considered potentially signficant contaminants in Dorchester Bay. However, it should be emphasized that they are presently not monitored by governmental authorities and appear to be less than experimental lethal values.

Some information is available on the concentrations of heavy metals in Dorchester Bay sediments. To our knowledge, there is no reliable information available to relate sediment heavy metals concentrations with toxicity to clams. However, signficant concentrations of heavy metals are present in Dorchester Bay sediment (Vol. II, Chapter III, Table III-3), and concentrations of heavy metals in soft-shell clams taken from Dorchester Bay appear to be generally higher than average values reported for Massachusetts coastal waters.

As with heavy metals, the data on other toxics is extremely limited. However, past analyses for the presence of Dieldrin-DDE and DDT in soft-shell clams, and winter flounder indicate that these pollutants are present in concentrations below FDA action levels.

Currently, no information on PCBs has been found for Dorchester Bay, the sediments, or indemic organisms.

In addition to the parameters discussed above, all waters of the Commonwealth must meet certain aesthetic parameters. Specifically, all waters must be free from pollutants of a nuisance nature (i.e., floating debris, scum, oil and grease, odor forming pollutants, etc.). While it is impossible to quantify such pollutants, there have been reports of floating debris, oil, and grease which have a deleterious effect on yacht club activities. In addition, odors have been reported in areas near the South Boston yacht clubs and around Tenean Beach. Additional information on these problems is presented later in this report.

## Criteria for Analysis of Existing Combined Sewers and Related Facilities

Data Sources. Most of the original plans and construction drawings for elements of the Boston Main Drainage System, dating from the early 1880's to the present, are on file with the Boston Water and Sewer Commission (BW&SC). These plans, especially the sectional plats of the South Boston and Dorchester areas, served as the major source of information regarding physical details of the system. The sectional plats illustrate all local laterals, trunk sewers, main interceptors, storm drains, and overflow conduits including their sizes, shapes and materials. The locations of manholes, and the ground surface and invert elevations (Boston City Base)* associated with each, are also shown on the plats. The plats have not been updated in detail since the 1940s' however, no major changes in the Dorchester Bay area system have occurred since that time.

*Boston City Base is 100-ft above MDC Base

The plans for the existing sewer system, prepared by Camp Dresser & McKee Inc. for the 1967 "Report on Improvements to the Boston Main Drainage System," were based on information obtained from these plats and serve as the base maps for this study.

In order to verify the location and/or sizes of sewers and system appurtenances, other sources were consulted or used as cross-references. The Metropolitan District Commission (MDC) provided plans of the conduits and facilities under their jurisdiction, such as the Columbus Park connection and headworks, Deer Island wastewater treatment plant, and rehabilitated tide gates and regulators. Plans of the Southeast Expressway and associated drainage and utilities in the surrounding areas were obtained from the Commonwealth of Massachusetts Department of Public Works.

Additional data were provided by past reports on the Dorchester Bay areas, including those by Process Research, Inc., [2], Charles A. Maguire & Associates [3] and Camp Dresser & McKee, Inc. [4,5] An important historical reference, the book Boston Main Drainage prepared by Clarke, [6] describes the design criteria, construction and operation of the original Boston main drainage system. Operational records from the Columbus Park headworks and the Calf Pasture pumping station were also valuable in determining existing system operation.

Field investigations were necessary to obtain additional information on system appurtenances, or to verify data which were conflicting or missing among the sources previously mentioned. Data collected in the field included descriptions and dimensions of all known regulator and tide gate chambers, depth-of-flow and velocity measurements in sewers and regulators at selected locations, and invert slopes of major sewer lines. (See Chapter V for a complete discussion of the field program.) A complete inventory of all known regulators and tide gates is presented in Appendix A.

Determination of Physical Characteristics of Existing Sewerage System - The physical characteristics of the existing sewerage system served as the data base for the computer models SEMSTORM and SWMM which were used both for analysis of the existing sewerage system and for simulating various potential CSO control methods. Accurate system data is necessary in order to assure good representation of system response during wet weather event simulations. Combined sewer modeling is discussed in detail in Chapter VII.

As discussed earlier, the BW&SC plats were the primary source of information regarding sewer invert and ground surface elevations. Elevations were verified in the field at all regulators and other key locations. Relative inverts were determined by measuring the distance from the manhole rim to the conduit invert and subtracting that difference from known ground elevations. If these values differed from those found on the plats, the field values were assumed to be correct.

Sewer sizes, shapes, and construction materials, also based on plans from BW&SC and other sources, are shown on the CDM base maps (see Figures A through E, appended). Field investigations were performed in order to verify any information that was considered questionable.

The Manning Formula was used to calculate conduit flow capacities as follows:

$$Q = \frac{1.49}{n} AR^{2/3} S^{1/2}$$

where Q is the mean flow in cubic feet/second, A is the cross-sectional area of flow in sq. ft., R is the hydraulic radius of the conduit in feet, S is the slope of the "energy line" (which is approximated by the invert slope of the conduit) and n is the coefficient of roughness of the conduit, specifically known as "Mannings n." In applying this formula, the greatest difficulty lies in the determination of the roughness coefficient, since there is no exact method of selecting the "n" value. To select a value of "n" actually means to estimate the resistance to flow in the channel; this is dependent upon many factors, including surface roughness (pipe material), conduit size and shape, and the presence of obstructions in the conduit. Thus the selection of Mannings "n" must be based on experience and engineering judgement.

For the capacity determinations, Manning's "n" values for existing conduits were chosen from textbook values [7] (given for sewers in fair to poor condition) which also agreed with the design criteria presented in past CDM reports. An "n" value of .015 was assumed for conduits of reinforced concrete and vitrified clay, and an "n" of .017 was assumed for those of brick, wood, stone, clay, plain concrete, iron and undetermined materials. In order to determine the actual friction factor in those conduits selected for flow monitoring, velocity and depth of flow measurements were made in the field. The "n" values were than calculated to yield an equivalent flow when the Manning formula was applied to that pipe, using the measured size, depth of flow, slope, and velocity.

Tributary drainage areas were delineated according to the direction of flow in the laterals or trunk sewers, determined by the invert elevations noted on the BW&SC plats. USGS contour maps were also used to further define and check all drainage sub-areas.

Field verification of the tributary areas was accomplished to some extent by flow measurement. Wet weather flows for a particular gaged rainfall event were calculated for each area based on size, population density, and land use, and compared to the gaged flows as a check of the area boundaries.

Design Criteria of Existing Facilities. According to Clarke, [6] the following design criteria were originally employed for the major intercepting sewers. The average dry weather flow was assumed to be 75 gal./day/cap. (or approximately 10 cu ft/day/cap.), with the maximum dry weather flow being 1.5 times the average. This was multiplied by the total population of the area served by the interceptor (based on an

assumed average population density over the area in question) to obtain the maximum daily dry weather flow. Added to this was a "provision for rain water" of 100 cu ft/sec (65 mgd), which was estimated to be the amount of runoff occurring from a 24-hour, 0.25-inch per hour intensity storm.

For example, the area served by the Boston main interceptor was assumed to have an average population density of 62.5 persons per acre, resulting in a total service population of 800,000. The total design flow would then equal 154.3 mgd or 238.88 cu ft/sec ((800,000 persons x 10 cu ft/day/cap. x 1.5) - (24 x 60 x 60) sec/day + 100 cu ft/sec.

The interceptors were sized to flow half full at a slope of 1 ft in 2,500 ft (0.0004 ft/ft), with a Manning's "n" value of .013. Although the Dorchester interceptor and the South Boston interceptor were built several years later than the Boston main interceptor, similar criteria were most likely applied, though this was not possible to verify.

For smaller sewers the method was slightly different. The same average dry weather flow as assumed but applied to an estimated future population per drainage area. The future dry weather flow was then doubled, and runoff from the 24-hour/.25-inch per hour intensity storm was added to this. The sewers were sized to flow half full, but if they were too small to be entered conveniently for cleaning, the sizes were increased (usually only in height) to allow sufficient head room for a city worker.

All sewers to be constructed in Boston are currently separated (i.e., separate sanitary sewers and storm drains) rather than combined. Consequently, storm drains and sanitary sewers each have their own set of design criteria, developed by the Boston Water and Sewer Commission.

For storm sewers, a design storm frequency of 10-25 years is used, depending on the location of the drainage area, and the rational formula is employed to compute runoff. Peak runoff rates are estimated by the rational formula equation $Q = CiA$, where Q is the peak runoff rate in cubic feet/second, C is a runoff coefficient, i is the average rainfall intensity (in./hr.) over a time equal to the time of concentration of the design storm event, and A is the size of the drainage area in acres. Reinforced concrete pipe is usually selected, and the conduit is designed to flow at a velocity of 3 to 12 ft/sec, with a Manning's "n" of .015.

Sanitary sewers are designed to serve a 30-year estimated future population, or at least 100 persons per acre. The dry weather flow is assumed to be 100 gal./day/cap. and an additional 500 gal./mile/day/inch diameter is added to allow for infiltration. The pipe is sized to carry 3 times this total design flow, flowing full. Clay pipe with an "n" value of .013 is usually used and the design velocities vary from 2 to 20 ft/sec.

## Design Criteria of Recommended Facilities

Design criteria for all facilities recommended in this facilities plan is presented in Chapter XII "Recommended Plan".

# CHAPTER IV - EXISTING FACILITIES

## General

The Dorchester Bay study area has a total area of 2,894 acres--2,446 acres in Dorchester and 448 acres in South Boston. The study area, including the major trunk sewers of the existing wastewater collection system and the delineation between the Dorchester and South Boston areas, is shown on Figures B, C, D, and E, appended.

## Combined Sewer Service Areas

Within the study area, there are eleven outlets that discharge combined sewer overflow to Dorchester Bay. Location of the outlets, along with a delineation of their tributary areas, is shown on the appended figures. In addition, the sub-areas within each tributary area that are served by either combined sewers or separate storm and sanitary sewers, and those unsewered areas that contribute stormwater runoff directly to receiving waters are also shown. A tabulation of the tributary areas, listing information on sizes (acreage), percent impervious, estimated existing population, and sewers for each area is shown in Table IV-1.

As shown in Table IV-1, it is estimated that 47.9 percent of the study area's total acreage is served by combined sewers, 35.9 percent is served by separate storm and sanitary sewers, and 16.2 percent of the area is coastal and unsewered, thus runoff flows overload directly to Dorchester Bay.

## Principal Interceptors, Combined Sewers, and Overflow Facilities

General. The Dorchester Bay study area is served primarily by three interceptors: (1) the Dorchester interceptor, which serves the Dorchester portion of the study area as well as the entire Neponset study area; (2) the south branch of the South Boston interceptor, which serves the South Boston portion of the study area; and (3) the trunk section of the South Boston interceptor, which serves the area between Dorchester and South Boston (referred to as the "middle area"). These facilities are owned and maintained by the City of Boston. A schematic of the existing sewerage system is shown on Figure IV-1.

Wastewater flow from the Inner Harbor study area that is tributary to the Columbus Park headworks enters Dorchester Bay's middle area through the Boston main interceptor and the north branch of the South Boston interceptor. The north and south branches of the South Boston interceptor join near Andrew square, to form the trunk section of the South Boston interceptor which conveys flow south to the Boston main interceptor near Columbus Circle. The Dorchester interceptor discharges its flow to the Boston main interceptor just downstream of the South Boston interceptor

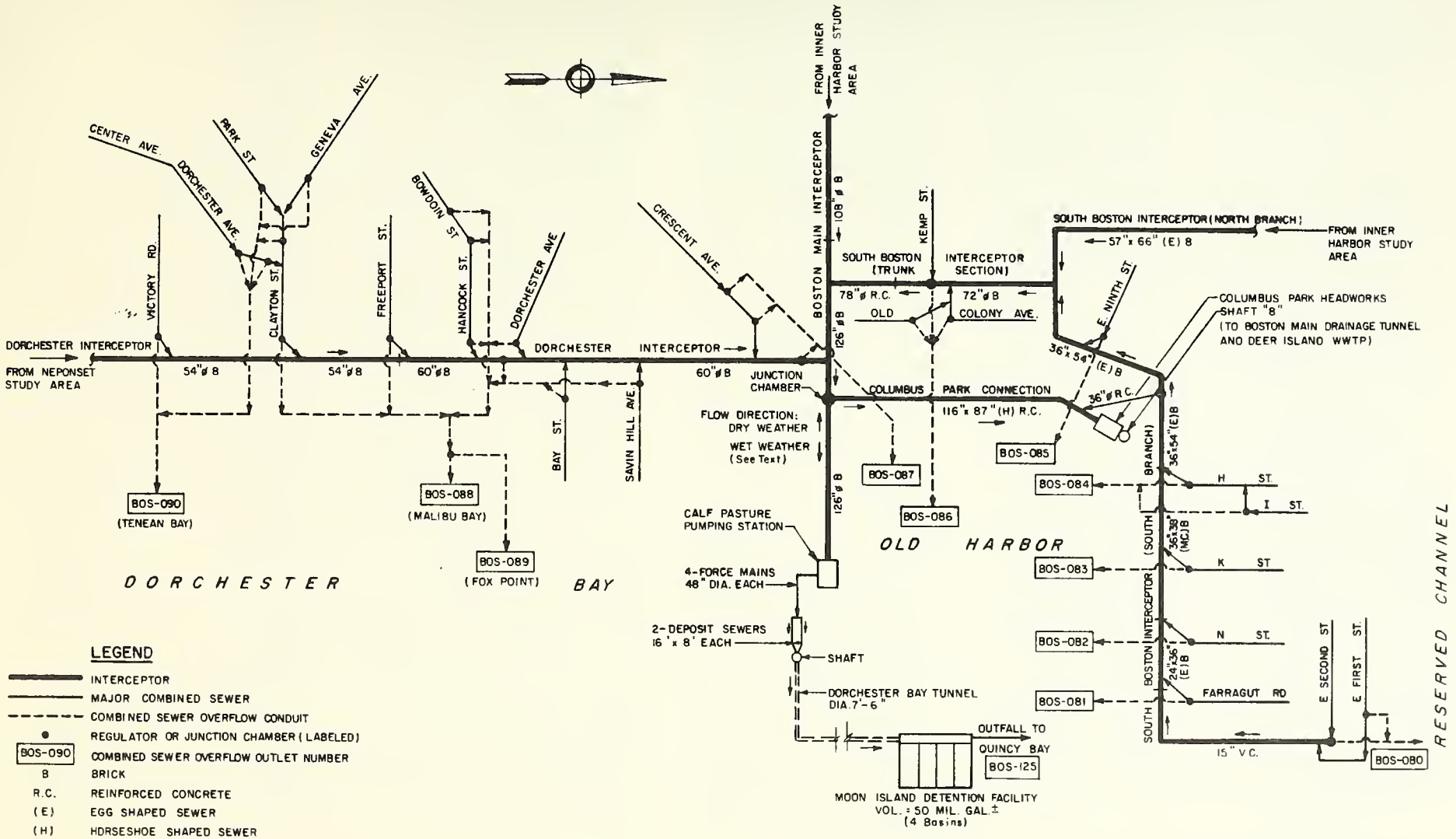
TABLE IV-1. INFORMATION ON DRAINAGE SUBAREAS[1]

| Subarea | Total Area, acres (Percent of Total Study Area) | Impervious Cover, % | Estimated Existing Population[2] | Area Served by Combined Sewers, acres (% of Subarea) | Area Served by Separated Sewers, acres (% of Subarea) | Direct Runoff Area, acres (% of Subarea) |
|---|---|---|---|---|---|---|
| BOS-080 | 47 (1.6%) | 75 | 1,713 | 47 (100%) | 0 | 0 |
| BOS-081 | 39 (1.3%) | 80 | 1,745 | 31 (79.5%) | 3 ( 7.7%) | 5 (12.8%) |
| BOS-082 | 82 (2.8%) | 90 | 4,663 | 60 (73.2%) | 12 (14.6%) | 10 (12.2%) |
| BOS-083 | 29 (1.0%) | 90 | 2,242 | 29 (100%) | 0 | 0 |
| BOS-084 | 64 (2.2%) | 85 | 3,517 | 53 (82.8%) | 0 | 11 (17.2%) |
| BOS-085 | 72 (2.5%) | 50 | 3,351 | 47 (65.3%) | 0 | 25 (34.7%) |
| BOS-086 | 152 (5.2%) | 45 | 4,725 | 82 (53.9%) | 24 (15.8%) | 46 (30.3%) |
| BOS-087 | 595 (20.6%) | 40 | 14,822 | 174 (29.2%) | 206 (34.6%) | 215 (36.2%) |
| BOS-089/088[3] | 539 (18.6%) | 80 | 13,539 | 310 (57.5%) | 163 (30.2%) | 66 (12.3%) |
| BOS-090 | 1,275 (44.2%) | 65 | 42,311 | 555 (43.5%) | 631 (49.5%) | 89 ( 7.0%) |
| Total | 2,894 (100%) | 63 | 92,628 | 1,388 (47.9%) | 1,039 (35.9%) | 467 (16.2%) |

[1] Refer to Figures B through D, appended for drainage subarea delineations. Drainage subareas are designated by their respective combined sewer overflow outlet number.

[2] Based on 1975 population data from the Central Transportation Planning Staff of the MAPC, and estimated to be essentially equal to existing (1979) population.

[3] CSO outlet BOS-088 provides hydraulic relief to outlet BOS-089, thus both outlets have the same drainage subarea (BOS-089). See text for further description.




## LEGEND

- INTERCEPTOR
- MAJOR COMBINED SEWER
- COMBINED SEWER OVERFLOW CONDUIT
- REGULATOR OR JUNCTION CHAMBER (LABELED)
- BOS-090 — COMBINED SEWER OVERFLOW OUTLET NUMBER
- B — BRICK
- R.C. — REINFORCED CONCRETE
- (E) — EGG SHAPED SEWER
- (H) — HORSESHOE SHAPED SEWER
- (M.C.) — MODIFIED CIRCLE SHAPED SEWER
- ø — CIRCULAR SEWER

FIG. IV-1 SCHEMATIC OF EXISTING SEWERAGE SYSTEM SHOWING MAJOR COMBINED SEWERS

COMMONWEALTH OF MASSACHUSETTS
METROPOLITAN DISTRICT COMMISSION
COMBINED SEWER OVERFLOW PROJECT
DORCHESTER BAY AREA FACILITIES PLAN

FIG. IV-1

connection. The Boston main interceptor carries flow from the entire Dorchester Bay and Neponset study areas, from a portion of the Inner Harbor study area, and from a portion of the Charles River Basin study area (during certain wet weather events only), east to the Columbus Park connection at Kosciuszko Circle, then north by way of the connection to the Columbus Park headworks. At the headworks, the wastewater passes through fine screens and grit chambers, prior to entering the Boston main drainage tunnel through Shaft "B," for transport to the Deer Island wastewater treatment plant.

Some stormwater relief for the existing system during certain rainfall events, is provided by the former Boston main drainage system outlet works which comprises the Calf Pasture pumping station, the Dorchester Bay tunnel, and the Moon Island detention facility. Operation of this complex reduces the volume of untreated overflows into near-shore Bay waters. Other system appurtenances include regulators, tide gates, and combined sewer overflow (CSO) outlets. A discussion of these appurtenances, the facilities metioned above, and the overall operation of the existing combined sewer system follows.

Combined Sewer System Operation. As discussed in Chapter III, combined sewers in the Dorchester Bay study area were designed to accept both dry weather wastewater flows and stormwater runoff. As shown on Figure IV-2, the existing system configuration is such that lateral sewers convey flow to trunk sewers which eventually discharge into an interceptor. One or more regulating devices are located on each trunk sewer. These serve to prevent surcharging of the interceptor during a rainfall event, by allowing flow in excess of interceptor capacity to overflow into conduits for eventual discharge to Dorchester Bay. This combined sewer overflow (CSO) is a mixture of stormwater runoff and normal dry weather wastewater flow.

There are three basic types of regulators within the study area. Each type is a static device, utilizing either an overflow weir, a sump, and/or an orifice. Each type will be described in detail later in this chapter. A tabulation of all system regulators is included in Appendix A. One or more tide gates are located on each CSO conduit, and serve to prevent the inflow of tidewater to the wastewater collection system during periods of high tide. Virtually all consist of a flap gate hinged at the top, which opens when the water level on the upstream side (i.e., within the CSO conduit) is greater than the tide level. A detailed discussion of tide gates, and the recent rehabilitation program conducted by the MDC, is presented later in this chapter. A tabulation of all system tide gates is also included in Appendix A.

Principal Interceptors. The Dorchester interceptor serves all of the 800-acre Neponset study area and 2,312 acres in the Dorchester portion of the Dorchester Bay study area, for a total tributary area of 3,112 acres. Constructed of brick, the northern 60-in. diameter portion has an estimated flowing full capacity of 28.2 mgd.

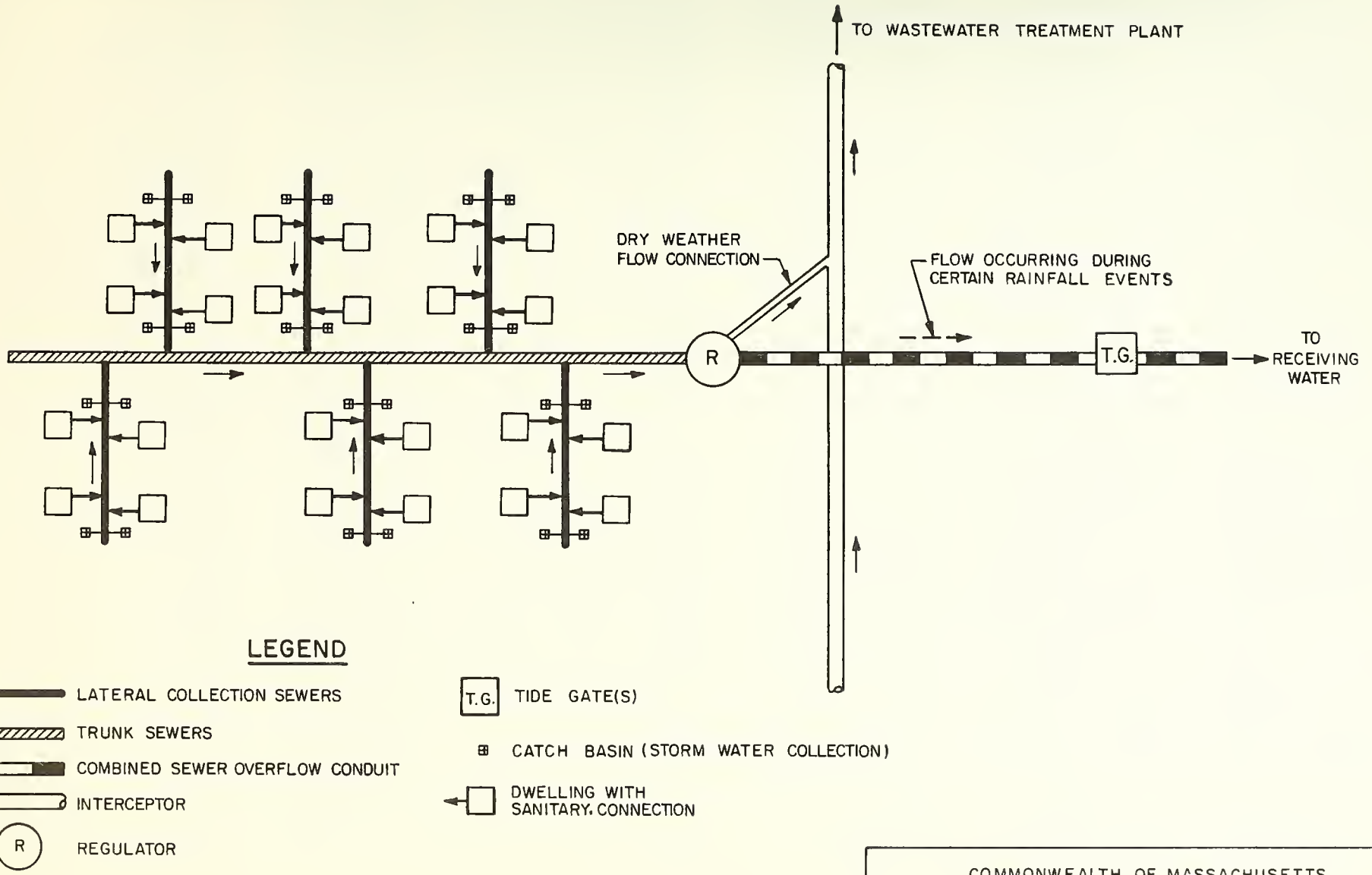



FIG. IV-2 SCHEMATIC SHOWING COMBINED SEWER SYSTEM OPERATION

COMMONWEALTH OF MASSACHUSETTS
METROPOLITAN DISTRICT COMMISSION
COMBINED SEWER OVERFLOW PROJECT
DORCHESTER BAY AREA FACILITIES PLAN

As indicated on Figures B, C, and D, appended, the interceptor begins in the Neponset study area as a 54-in. diameter sewer at Granite Avenue south of Hilltop Street. At this point, it receives the dry weather flow from the Metropolitan District Commission's Neponset River Valley low level sewer. The interceptor then proceeds generally northward, east of Dorchester Avenue, to Orchardfield Street, where it increases to a 60-in. diameter brick sewer. From this location, it travels northward until it connects with the main interceptor of the Boston main drainage system near Kosciuszko Circle.

There are two regulators along the 3.7-mile Dorchester interceptor. The first is a side overflow weir-type regulator, located on private land near Hoyt Street. Overflows from this regulator are tributary to CSO outlets BOS-088 and BOS-089* (Figure C, appended). The second regulator is a high outlet-type, located at the connection with the Boston main interceptor near Kosciuszko Circle. Overflows from this location discharge to Old Harbor, through outlet BOS-087 (Figure B, appended).

Wastewater from South Boston is collected by two branches of the South Boston interceptor. The drainage divide between the tributary areas of these branches is located approximately at Dorchester and East Fourth streets, as shown on Figure B, appended. The north branch is considered part of the Inner Harbor study area and is not addressed here. The south branch serves the South Boston portion of the Dorchester Bay study area and has a tributary area of 302 acres. Varying from a 15-in. diameter sewer to 36" x 54" egg-shaped sewer, the south branch has a flowing full capacity of about 16.8 mgd at its intersection with the north branch.

As indicated on Figure B, appended, the South Boston interceptor (south branch) begins as a 15-in. diameter sewer at the regulator located at the intersection of Farragut Road and East Second Street. Here, it extends south to East Sixth Street, then west to O Street. At O Street, the size increases to 24 in. x 36 in. (E)** and extends south to East Eighth Sreet, east to N Street and south to Marine Road, where the size increases to 36 in. x 38 in. (MC).*** The interceptor then extends west in Marine Road to Columbia Road, where the size again increases to 36 in. x 54 in. (E)** at H Street. In Columbia Road, the interceptor continues west to Devine Way, where it joins with the north branch of the South Boston interceptor near Logan Way to form the trunk section of the South Boston interceptor.

There is one regulator, along the 2.1 mile long south branch, located in Columbia Road between Old Harbor and Covington streets. It is a side overflow weir-type regulator with a direct, 36-in. diameter connection to the Columbus Park connection just before it enters the Columbus Park headworks. This regulator helps to prevent surcharge conditions in the interceptor during periods of high flow (i.e., during rainfall events).

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*Outlet BOS-088 serves a hydraulic relief for outlet BOS-089 during a coincident high intensity rainfall event and high tide. This will be explained in detail later in this chapter.

**(E) = egg-shaped sewer.

***(MC) = modified circle-shaped sewer.

As discussed in the preceding section and as shown on Figure IV-1 and Figure B, appended, the north and south branches of the South Boston interceptor join at Devine Way to form the trunk section of the South Boston interceptor. The trunk section begins as a 72-in. diameter brick conduit extending south in easement beneath the MBTA's Red Line tracks to Locust Street, where the sewer increases to a 78-in. diameter reinforced concrete conduit. From this location, it runs parallel with the Southeast Expressway to its connection with the Boston main interceptor at Mt. Vernon Street. The South Boston interceptor is about 2,700 ft long and has an estimated flowing full capacity of 65.6 mgd in the 78-in. diameter section. It serves a total tributary area of about 1,022 acres, including about 574 acres from the portion of the Inner Harbor study area that is tributary to the north branch. There is one regulator located on the interceptor near Sterling Square (Figure B, appended). It consists of a high outlet pipe which allows the interceptor to overflow to waters off of Carson beach through CSO outlet BOS-086.

As shown on Figure B, appended, the Boston main interceptor enters the Dorchester Bay study area at the intersection of Boston and Mt. Vernon streets. At this point, the interceptor is a 108-in. diameter brick conduit which conveys dry weather flow from a portion of the Inner Harbor study area. Going east in Mt. Vernon Street to Columbia Road, the size increases to 126 in. in diameter. From there, the interceptor extends east in Mt. Vernon Street and accepts flow from the trunk section of the South Boston interceptor near Columbia Circle and from the Dorchester interceptor in the vicinity of Kosciuszko Circle. Prior to the completion of the Columbus Park connection and Columbus Park headworks in 1968, the Boston main interceptor conveyed flow east in Mt. Vernon Street to the Calf Pasture pumping station located on Columbia Point. All flow was then pumped, through the Dorchester Bay tunnel, to the Moon Island detention tanks, where it was released into Quincy Bay untreated with the outgoing tide. In order to direct flow from the Boston main inteceptor to the Columbus Park connection and, eventually to Deer Island, a junction chamber was built as part of the Columbus Park connection construction contract. At the junction chamber, located near Kosciuszko Circle, the Boston main interceptor has an estimated flowing full capacity of 185 mgd and serves a total tributary area of about 10,220 acres.

The junction chamber consists of a 45-ft long overflow weir with its crest about 6.2 ft above the invert, adjustable within a limited range (about 2 in.), and a 4'-0"-wide opening for stop logs located at the downstream end of the weir. Field inspection of the chamber in October 1978 revealed that no stop logs were in place. Thus, the section of the Boston main interceptor between the junction chamber and Calf Pasture pumping station and the Columbus Park connection is hydraulically connected.

As indicated on Figures B and C, appended, wastewater discharging from the Columbia Point area, which includes the Columbia Point housing Project and the University of Massachsuetts, enters the Boston main interceptor along Mt. Vernon Street. For flow to enter the Columbus Park

connection through the 4-ft wide stop log opening, it must flow in the reverse direction of the invert slope. Flow in this direction will only occur when sufficient head is built up in the interceptor to allow flow into the connection. It should be noted that if stop logs were in place in the junction chamber, the wastewater level in this section of the Boston main inteceptor would have to be greater than the weir crest elevation at the junction chamber, before flow would discharge over the weir and into the connection. This condition results in excessive solids deposition in this section of the Boston main interceptor since the velocity of flow is close to zero. During activation of the Calf Pasture pumping station, excessive flow in the Boston main interceptor, as it approaches the connection, discharges over the weir and through the 4-ft stop log opening at the junction chamber and flows eastward in the Boston main interceptor to the Calf Pasture pumping station for discharge to Moon Island.

Columbus Park Connection. As shown on Figure B, appended, the 116-in. by 87-in. horseshoe-shaped, reinforced concrete Columbus Park connection conduit, owned and maintained by the MDC, extends northward about 3,500 ft from the junction chamber at the Boston main interceptor in Mt. Vernon Street to the Columbus Park headworks. The estimated capacity of the connection, under the operating condition with the water surface at the entrance to the headworks at El. 90.3 (MDC base), and at the level of the weir crest at the junction chamber (El. 93.2), is estimated to be 180 mgd. With a water surface elevation greater than junction chamber the weir crest, additional flow is capable of passing through the connection conduit. When the connection conduit is flowing full, and the hydraulic slope is parallel to the invert slope, the capacity is estimated to be about 135 mgd.

Columbus Park Headworks. The headworks at Columbus Park is one of three tributary to the Deer Island wastewater treatment plant. All are owned and operated by the MDC. The Chelsea Creek headworks and Ward Street headworks serve the Inner Harbor and Charles River Basin study areas and some areas outside of the MDC project area boundary, thus will not be discussed herein. As shown on Figure IV-3, the Columbus Park headworks is comprised of four separate channels. Manually-controlled sluice gates at the influent of each channel allow any number or combination of channels to be utilized at any given time, while providing the headworks with flow throttling capability. Each channel was equipped with both coarse and fine screens; however, the coarse screens, which consisted of 3/4-in. x 3-in. bars with 3-1/2-in. openings, were removed in 1970, due to low screenings accumulation. The fine screens consist of 5/8-in. x 2-1/2-in. bars with 3/4-in. openings, and they are mechanically cleaned.

After screening, flow proceeds to grit chambers equipped with chain flight collectors. The flights direct grit to a sump where a screw conveyor transfers the grit to an ejector for transport to a grit hopper. Screenings from the fine screens are also ejected to a screenings hopper. Grit and screenings from the hoppers are discharged to a dump truck for transport to Deer Island for disposal with the wastewater treatment plant sludge. Headworks personnel report that approximately 30 cu ft of grit and screenings are produced each day.

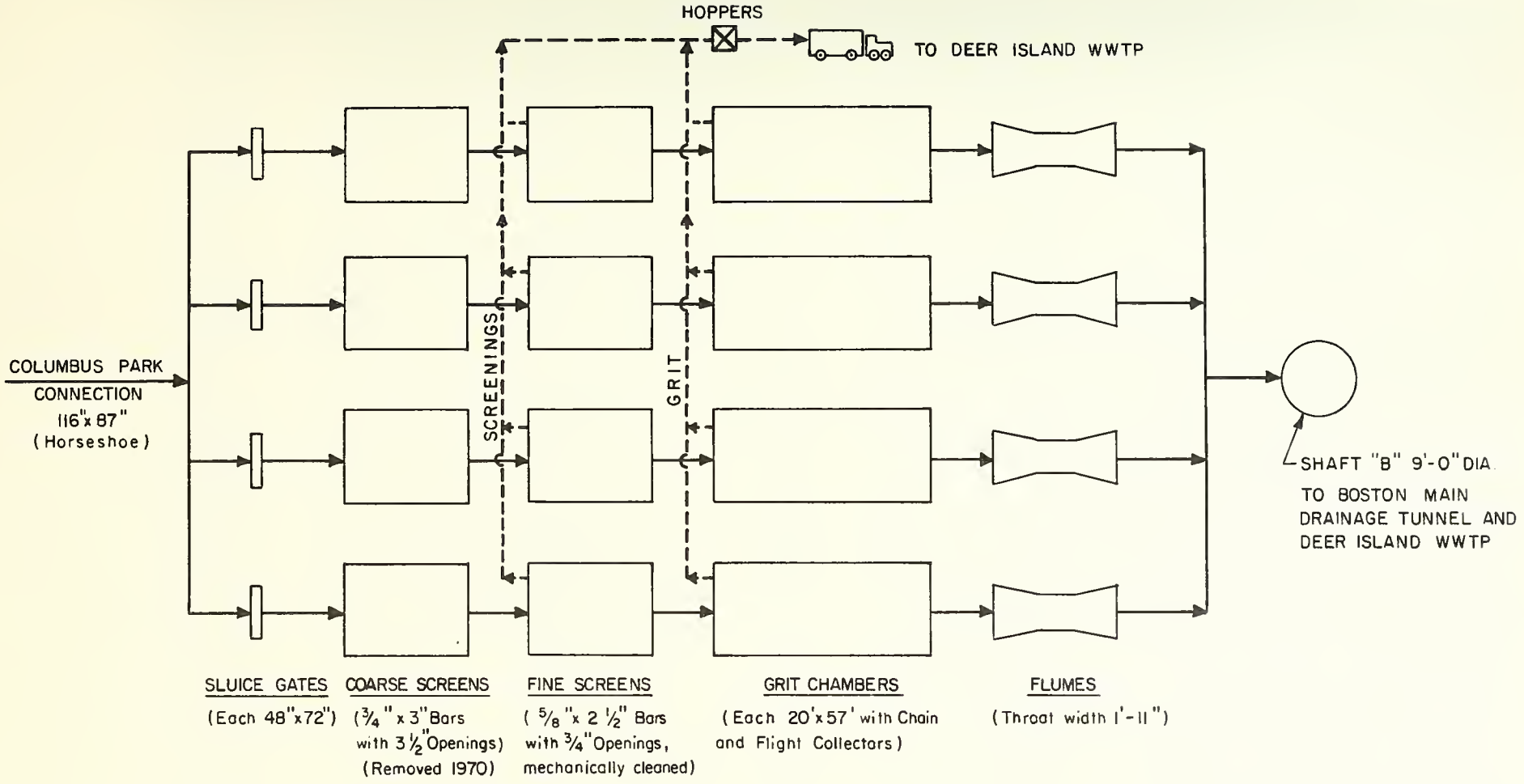



FIG. IV-3 SCHEMATIC OF COLUMBUS PARK HEADWORKS

COMMONWEALTH OF MASSACHUSETTS
METROPOLITAN DISTRICT COMMISSION
COMBINED SEWER OVERFLOW PROJECT
DORCHESTER BAY AREA FACILITIES PLAN

Following the grit chambers, flow proceeds through measuring flumes (throat width of 23 in.), and then enters the effluent channel for conveyance to the 9-ft 0-in. diameter Shaft "B". The shaft extends downward approximately 300 ft to the Boston main drainage tunnel (MDC Base El. = -200). The tunnel, which begins at Shaft "A" located at the Ward Street headworks, extends about 13,800 ft (10 ft diameter) to Shaft "B", then about 23,800 ft (11 1/2 ft diameter) to Shaft "C" at the Deer Island pumping station. With the pumping station operating at its maximum rated capacity of 850 mgd, the carrying capacity of the Boston main drainage tunnel is estimated to be about 440 mgd, operating under a surcharged condition. The remaining pumping station capacity is required for pumping flows from the North Metropolitan relief tunnel which receives flow form the Chelsea Creek headworks. Deer Island also treats flow from the Winthrop terminal sewer, however, this waste stream enters the plant by gravity, downstream of the pumping station.

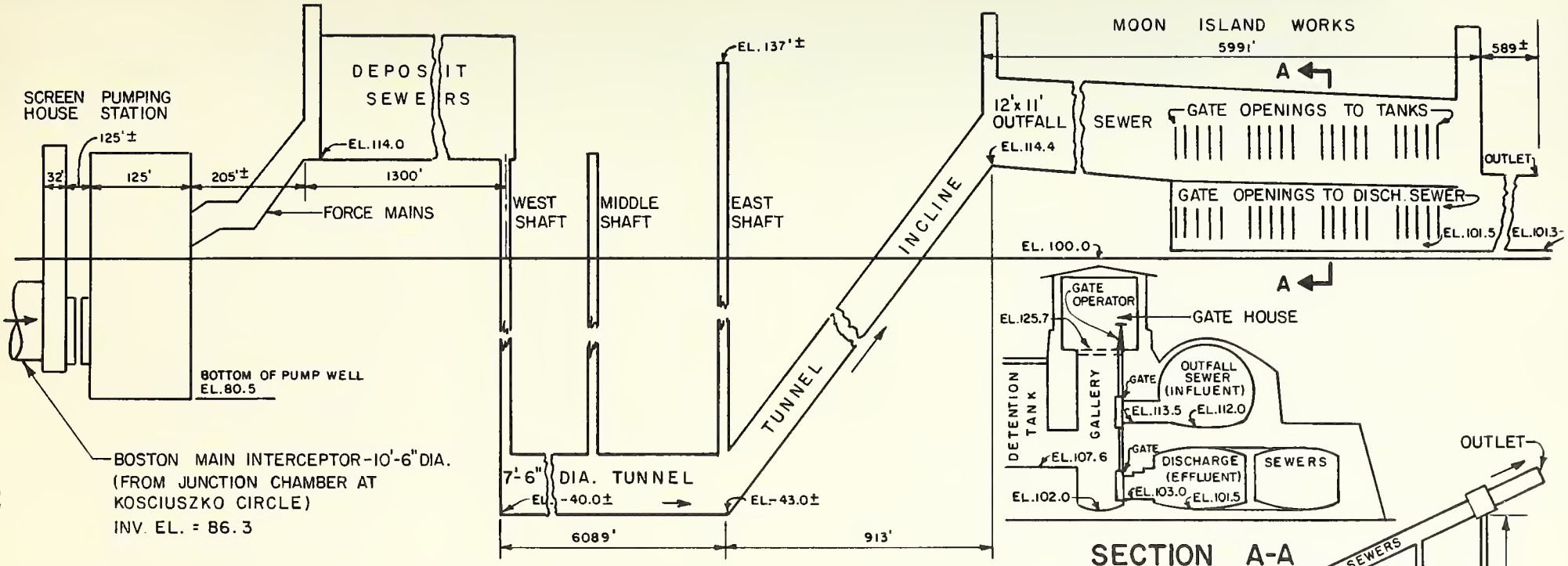
The maximum hydraulic capacity of the Columbus Park headworks is estimated to be 180 mgd. The current average daily flow rate is about 65 mgd. If, during a large rainfall event, the total flow through the three headworks should approach the maximum pumping capacity at Deer Island (850 mgd), throttling at one, two, or possibly all three headworks is often required. Throttling at the Columbus Park headworks is accomplished by the closing of the sluice gates located at the influent of each channel. This action results in an increase in the depth of flow in the Columbus Park connection, and also in the Boston main and South Boston interceptors.*

If an overflow occurs or if one is anticipated at the junction chamber at Kosciuszko Circle, the Calf Pasture pumping station is alerted and pumping to Moon Island is initiated.

Calf Pasture - Moon Island Facilities. The Calf Pasture - Moon Island facilities, owned and operated by the City of Boston, provide hydraulic relief to the Columbus Park headworks and its tributary interceptors. If such a relief mechanism were not available during high flows (i.e., during certain rainfall events), wastewater in the Boston main interceptor and in the South Boston interceptor would surcharge and eventually discharge through the Dorchester Brook conduit and the Roxbury canal to Fort Point channel, as well as through existing CSO outlets.

The facilities include the Calf Pasture pumping station, the Dorchester Bay tunnel, and the Moon Island detention facility. These comprise the former Boston main drainage outlet works, and were in continuous use until completion of the Deer island wastewater treatment plant in 1968. All of the facilities were built in the early 1880's and are, because of age and lack of maintenance, in poor condition.

*Based on a flow of 180 mgd at the headworks, hydraulic calculations conclude that the Dorchester interceptor would not be affected by this backwater condition due to its relatively high invert elevation at its connection to the Boston main interceptor

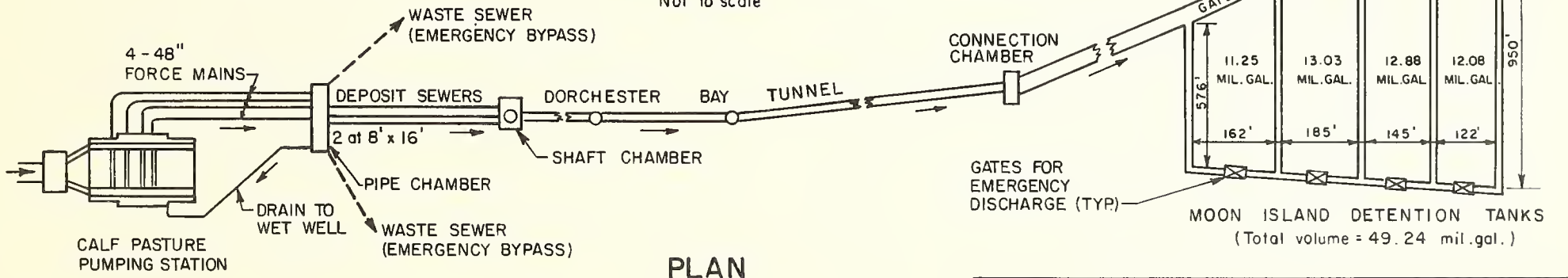



## ELEVATION

Not to scale

## SECTION A-A

Not to scale




## PLAN

Not to scale

NOTE: ALL ELEVATIONS REFER TO MDC BASE.

# FIG. IV-4 CALF PASTURE-MOON ISLAND FACILITIES

COMMONWEALTH OF MASSACHUSETTS

METROPOLITAN DISTRICT COMMISSION

COMBINED SEWER OVERFLOW PROJECT

DORCHESTER BAY AREA FACILITIES PLAN

As shown on Figure IV-4, and on Figures B, C, and E, appended, flow from the Boston main interceptor in Mt. Vernon Street enters a screen house, where the flow divides into two channels.

Each channel is equipped with two "cages" arranged in series, each consisting of 3/4-in. round iron rods which serve as screens. For cleaning, the "cages" are hoisted to the main floor and debris is removed manually. Field investigations revealed however, that this screening mechanism is no longer operational due to equipment deterioration. Flow then enters the wet well of the pumping station, where it is pumped through four 48-in. diameter force mains to a pipe chamber located about 200 ft east of the pumping station. Currently, three out of a total of six possible pumping units are in operating condition, each reported to be rated at about 32.8 mgd. The wastewater is lifted vertically about 35 ft to the pipe chamber, where it enters two, 8-ft wide x 16-ft high deposit sewers. These 1,300-ft long conduits extend to the west shaft of the Dorchester Bay tunnel and are dammed at the east end, in order to maintain a constant depth of about 11.5 ft. They were originally designed to serve as sedimentation tanks for the purpose of preventing large solids and grit from entering the tunnel. At the west shaft chamber, there are two 6.5-ft diameter "waste sewers" which serve as an emergency overflow in the event that the Dorchester Bay tunnel requires maintenance or if a failure develops. The waste sewers discharge to an outlet, built through the sea wall, located at the end of the pier. However, from discussions with Boston Water & Sewer Commission, these sewers have not been employed since the late 1960's when dry weather flow began to be routed to Deer Island.

Flow from the deposit sewers enters the 7.5-ft diameter west shaft which connects to the brick-lined 7.5-ft diameter Dorchester Bay tunnel. The tunnel was built in rock at an invert elevation averaging about -42 ft (MDC Base) for 6,089 ft, then inclines upward for about 913 ft to the outfall sewer located in the Squantum section of Quincy. At this point, the tunnel's invert elevation is 114.4 ft (MDC Base). The outfall sewer is a 12-ft x 11-ft horseshoe-shaped brick conduit extending about 6,000 ft to the Moon Island detention facility. In addition to the west influent shaft, a middle shaft was built on the main section of the tunnel for construction purposes, and an east shaft was built where the tunnel begins its incline (Figure E, appended). The middle shaft was sealed off after completion of the tunnel. The east shaft remains accessible and is used for conducting flow-capacity tests and for maintenance purposes.

The Dorchester Bay tunnel was completed on 25 July 1883. At that time, flow-capacity tests indicated that the tunnel capacity was about 200 mgd, based on a Manning's "n" of 0.013 (equivalent Hazen-Williams C Value of approximately 130). Tests conducted in 1946 by the City of Boston Sewer Division indicated that the carrying capacity was about 134 mgd, and that the average Hazen-Williams C Value was 73. Additional tests were conducted in 1947 by Charles A. Maguire & Associates, after removing a significant amount of debris from the east shaft. These tests indicated the carrying capacity to be about 154 mgd at a C Value of about 92. Leakage tests also conducted in 1947 indicated that the tunnel was fairly tight at that time, with negligible leakage under normal operating conditions. The current capacity and structural condition of the tunnel is unknown.

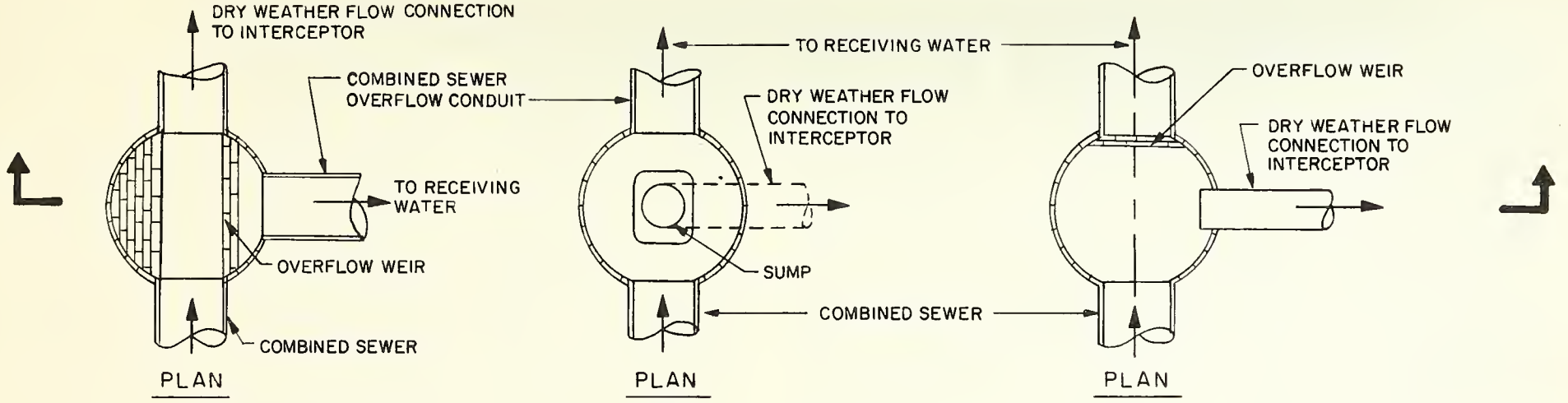
The Moon Island detention facility consists of four basins with a total capacity of about 49.24 mil gal based on a water surface elevation of 125.0 ft. (MDC Base). As indicated on Figure IV-4, flow enters the detention tanks through sluice gate openings located at an invert elevation of 113.5 ft (MDC Base). The tanks are drained into the discharge sewers on the outgoing tide by opening gates located at an invert elevation of 103.0 ft. Field investigations, however, revealed that all sluice gates are presently inoperable, thus flow control is no longer possible. The facility's outlet sewers consist of two 10.8-ft x 12-ft wide, horseshoe-shaped brick conduits which extend about 600 ft from the tanks to the receiving waters of Quincy Bay.

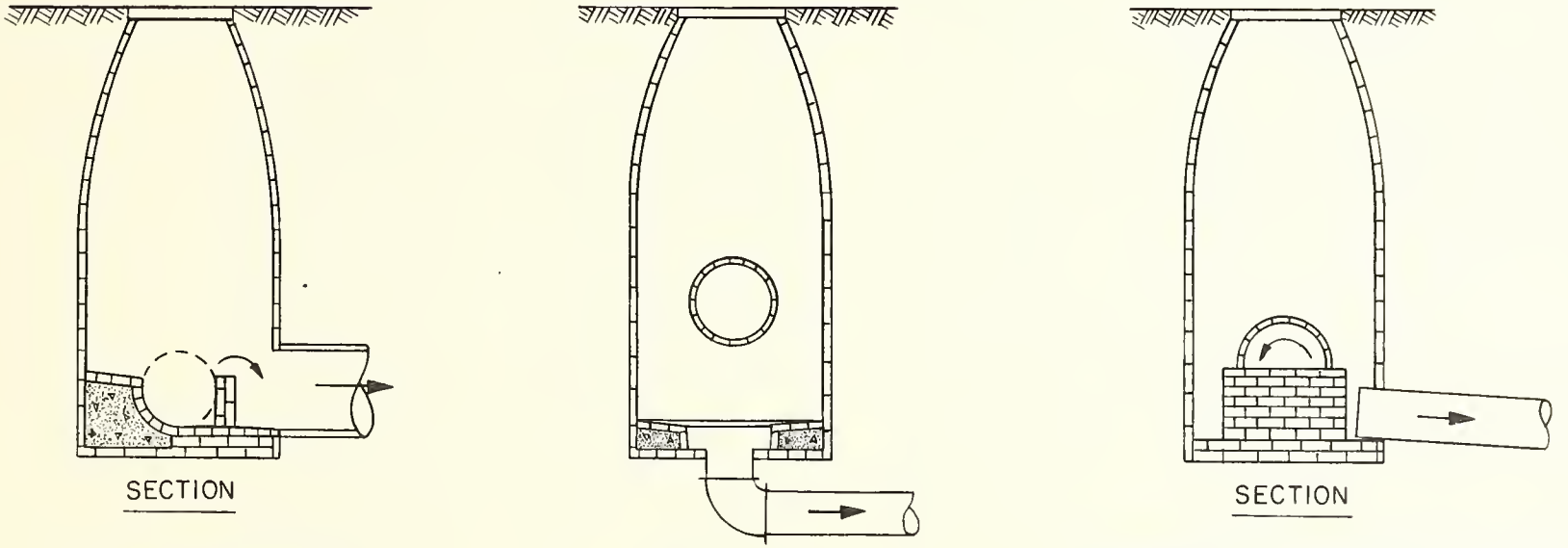
Based on operating records obtained from the Boston Water and Sewer Commission, the Calf Pasture - Moon Island facilities have been activated an average of 17 days per year since 1971. The total annual volume pumped averages about 258 mil gal. The quality and quantity of discharge from the Moon Island Detention Facility (NPDES Outlet No. 125) is monitored and recorded by the Boston Water and Sewer Commission in accordance with NPDES permit requirements. During each discharge, flows are recorded at the Calf Pasture Pumping Station and composite grab samples are taken at the outlet at Moon Island for determination of $BOD_5$ and TSS. The EPA is notified after each overflow event that quality and quantity measurements have been made; a second report is submitted to the EPA when the results of the sample analysis are available.

Regulators. There are 34 known regulators within the Dorchester Bay study area. They are all static-type flow control devices that utilize weirs, sumps, and/or high outlet pipes for flow regulation. Their purpose is to prevent surcharging of the interceptor by allowing flows in excess of interceptor capacity to be diverted and subsequently discharged to another point in the interceptor system or to Dorchester Bay. All are owned and maintained by the City of Boston.

The three basic types of regulators within the study area are shown on Figure IV-5. They are: (1) side overflow weir, (2) sump with high outlet, and (3) weir with orifice connection. In a side overflow weir-type regulator, overflow of combined wastewater occurs when the flow level of the combined sewer is greater than the overflow weir elevation. The sump with a high outlet-type uses the submerged orifice principle to regulate flow to the interceptor. When the head or flow level over the orifice is greater than the invert elevation of the overflow conduit, flow is discharged to the receiving waters through the conduit, and the amount of flow going to the interceptor remains somewhat constant. The orifice connection is sized to pass a flow rate that would not surcharge the interceptor. The weir with a side outlet orifice connection-type operates by the same principle, only the head over the orifice is controlled by a weir. Some of the regulators within the study area vary slightly in configuration from the general types shown on Figure IV-5; however, all utilize the same principles of static flow regulation.

An inventory of all study area regulators, including location, type, and observed conditions, is presented in Appendix A.




SIDE OVERFLOW WEIR

SUMP WITH HIGH OUTLET

WEIR WITH SIDE OUTLET ORIFICE

FIG. IV-5 TYPES OF REGULATORS WITHIN DORCHESTER BAY STUDY AREA

COMMONWEALTH OF MASSACHUSETTS
METROPOLITAN DISTRICT COMMISSION
COMBINED SEWER OVERFLOW PROJECT
DORCHESTER BAY AREA FACILITIES PLAN

Tide Gates. There are 28 tide gate chambers within the Dorchester Bay study area. Their purpose is to prevent seawater inflow into the wastewater collection system. Most are metal flap gates hinged at the top; however, some are old wooden gates that are either hinged at the top or at the sides. Most tide gates inspected during the summer of 1978 were found to be in good working condition. All are owned and maintained by the City of Boston.

The Metropolitan District Commission (MDC) conducted a Tide Gate Rehabilitation Program for the City of Boston between 1970 and 1977. Under this program, the MDC was given jurisdiction over many of the tide gates within the city. During the program, the MDC maintained the gates, performed weekly inspections, and noted problem areas. The program resulted in rehabilitation of many of the tide gates within the study area. Construction was completed in May 1977, and jurisdiction of all tide gates was returned to the City of Boston on 1 December 1977. A summary of the tide gates that were rehabilitated or replaced in the Dorchester Bay study area under this program is included in Appendix A. An inventory of all tide gates within the study area, based on field investigations performed during the summer of 1978, is also included in Appendix A.

Combined Sewer Overflow (CSO) Outlets. There are 11 CSO outlets within the Dorchester Bay study area. Their locations are shown on Figures B and C, appended, with the outlet numbers representing those appearing in the current City of Boston's National Pollutant Discharge Elimination System (NPDES) permit. As shown, each number is prefixed by "BOS" in order to delineate those outlets within all four study areas that are located within the City of Boston. A description of each outlet is given in Table IV-2.

As indicated on Figure B and C, appended, several outlets discharge into waters adjacent to public beaches. Seven (No's. BOS-081 through 087) discharge into Old Harbor, where Carson Beach and L Street Beach are located. Outlet BOS-088 discharges into Malibu Bay, the site of Malibu Beach, and outlet BOS-090 is located in Tenean Bay, just north of Tenean Beach. Outlet BOS-080 discharges into the Reserved Channel which is used as a shipping channel. As will be discussed later in this report, boating and harvesting of the area's shellfish beds may be affected by discharges of combined sewer overflows through these outlets. Several of the area's yacht clubs are also located adjacent to CSO outlets.

The outlet located in Malibu Bay (No. BOS-088) serves as an emergency relief for the system tributary to outlet BOS-089 at Fox Point. During certain rainfall events and a coincident high tide, the tide level will prevent the tide gates at Fox Point from opening, causing the wastewater level to rise in the CSO conduit located along Malibu Beach, upstream of the gates. As indicated on Figure C, appended, an overflow weir-type regulator is located on this CSO conduit between the MBTA tracks and the Southeast Expressway. When the level in the CSO conduit reaches the weir

TABLE IV-2. COMBINED SEWER OVERFLOW OUTLETS
WITHIN THE DORCHESTER BAY STUDY AREA

| CSO Outlet Number[1] | Location | Description[2] | Receiving Water |
|---|---|---|---|
| BOS-080 | Farragut Rd. (extended north) | 72" x 72" (U)C | Reserved Channel |
| BOS-081 | Farragut Rd. (extended south) | 30" (ø)B | Old Harbor |
| BOS-082 | N St. (extended) | 48" (ø)C.I. | Old Harbor |
| BOS-083 | K St. (extended) | 36" (ø)C.I. | Old Harbor |
| BOS-084 | H St. (extended) | 60" (ø)C.I. | Old Harbor |
| BOS-085 | Rev. R.A. Burke St. (extended) | 2 @ 60" (ø)C.I. | Old Harbor |
| BOS-086 | Kemp St. (extended) | 3 @ 60" (ø)C.I. | Old Harbor |
| BOS-087 | Near Columbia Circle | 2 @ 84" x 96" (R)R.C. | Old Harbor |
| BOS-088 | Malibu Beach | 2 @ 84" (ø) C.I. | Malibu Bay |
| BOS-089 | Fox Point | 2 @ 90" x 82" (R)R.C. | Dorchester Bay |
| BOS-090 | Commercial Point | 144" x 180" (R) R.C. | Tenean Bay |

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1. For locations, refer to Figs. B and C appended.

2. Nomenclature as follows:

(U) = U-shaped conduit
(ø) = Circular conduit
(R) = Rectangular conduit
C = Concrete
B = Brick
C.I. = Cast iron
R.C. = Reinforced concrete

crest elevation, overflow through outlet BOS-088 occurs. The purpose of this outlet is to prevent surcharging of the tributary sewers and subsequent flooding of tributary land areas.

Outlet BOS-090, located just south of Commercial Point (Figure C, appended), discharges into a man-made earthen enclosure, built as part of an abandoned landfill operation. Flow out of this enclosure is through a 36-in. diameter steel pipe discharging to Tenean Bay. Considering that the CSO outlet is a 12-ft x 15-ft conduit, this 36-in. diameter pipe represents a considerable hydraulic restriction in the system. In addition to reducing flow capacity, a backwater condition exists that reduces flow velocity in the CSO conduit, resulting in an excessive buildup of sediment. The area of primary concern, however, centers around the man-made earthen enclosure at the mouth of this outlet. This enclosure causes flow retention, and, thus, settling of solids. A considerable buildup of sludge has formed at the bottom of this enclosure, which is close to the Old Colony Yacht Club; the result is odors and generally objectionable conditions at the site. It also serves as an insect breeding ground during the summer months, and represents a potential health hazard to nearby areas.

## Performance of Existing Facilities

Overall Performance. Overall, the performance of the Dorchester Bay area wastewater collection system is good. Based on field investigations during the summer and fall of 1978, no surcharge conditions exist in any of the major interceptors, and all currently appear to convey flow at adequate velocities. The Columbus Park headworks appears to be in good working condition, with all solids removal equipment functioning properly. The Calf Pasture - Moon Island facilities are very old; and because of lack of maintenance, they are in relatively poor condition. Only three of six pumps are in operating condition at Calf Pasture. The current condition of the Dorchester Bay tunnel is unknown; however, as mentioned previously, it is expected that its carrying capacity is relatively low, due to continuous sediment build up. The Moon Island detention facility is also in poor condition, and as previously mentioned, the inlet and outlet sluice gates no longer operate.

No major structural deficiencies were discovered in the regulators, tide gates, and sewer manholes that were inspected during the course of this study. However, several maintenance-related problems were discovered, generally consisting of blockages within the sewers and regulators due to excessive sediment buildup. These maintenance problems and a small number of direct, dry weather connections to overflow conduits that were also discovered, result in several dry weather flow discharges to Dorchester Bay. Attempts were made by the Boston Water and Sewer Commission (BW&SC) and by CDM field personnel to clean all known blocked regulators and sewers, but in some cases the blockages have recurred.

Maintenance-Related Problems. Five maintenance-related problems were discovered during field investigations performed from July to November 1978. All were due to blockages within regulators and/or their dry weather flow connection to an interceptor. They are as follows:

1. First Street, between Farragut Road and P Street - South Boston (Figure B, appended)

A metal flap gate on the dry weather flow outlet from this regulator was found to be in the closed position, resulting in all dry weather flow tributary to this regulator discharging to the Reserved Channel through outlet BOS-080. The BW&SC was informed of this, and it is expected that the gate will be repaired in the near future.

2. Farragut Road near E. Sixth Street - South Boston (Figure B, appended)

Frequent complaints have been voiced by area residents concerning oil discharges from the 30-in. combined sewer overflow conduit located at Kelly's Landing (Outlet BOS-081). The problem is due to a blockage in the dry weather flow connection from the regulator at East Sixth Street and Farragut Road to the South Boston interceptor. Also, oil is entering the combined sewer in Farragut Road upstream of this regulator from surface drains located on, or adjacent to, the property of a local fuel oil company. These oil discharges have been observed by field personnel during the entire field program, thus they are not isolated incidents. The BW&SC was notified and they responded by sending a cleaning crew to free the dry weather flow connection. The regulator and the dry weather flow connection were cleaned to an extent that allowed dry weather flow to enter the interceptor; however, the blockage has recurred and the problem persists. It appears that a thorough cleaning of the regulator, the dry weather flow connection, and the interceptor just downstream of the dry weather flow connection is required to prevent further dry weather discharges at this location. Also, oil discharges to the combined sewer in Farragut Road should be eliminated.

3. Savin Hill Avenue at Auckland Street - Dorchester (Figure C, appended)

It was discovered that the regulator at this location was directing dry weather flow to the combined sewer overflow conduit in Auckland Street, due to a blocked dry weather flow connection. This flow eventually discharged to Dorchester Bay at Fox Point through outlet BOS-089. The BW&SC was notified, and a work crew cleaned the regulator and the dry weather flow connection, thus allowing dry weather flow to enter the interceptor.

4. Bay Street at Auckland Street - Dorchester
(Figure C, appended)

Field investigations at this location revealed what appeared to be a direct dry weather flow connection to the combined sewer overflow conduit in Auckland Street. Considering that the lack of a regulator at this location does not conform with the system's conceptual design, additional field investigations were performed. A manhole on the 28-in. x 42-in. combined sewer in Bay Street near Auckland Street was entered by the field crew and the bottom deposits were removed by hand. A blocked dry weather flow connection to the Dorchester interceptor was found and subsequently cleaned by the field crew. It was evident that this sump-type regulator has been blocked for several years, due to the extremely septic condition of the bottom deposits that were removed. Although the regulator may now be functioning properly, debris readily accumulates in the manhole and frequent cleaning is required. This is possibly due to an undersized dry weather connection pipe causing buildup of flow and debris in the regulator. Also, the dry weather flow outlet is located at the bottom of the regulator in a sump which is easily blocked by debris contained in the flow. (This regulator is a sump with high outlet-type, as shown on Figure IV-5.)

5. Leroy Street near Ditson Street - Dorchester
(Figure C, appended)

Field investigations revealed an improperly functioning regulator at this location. This resulted in dry weather flow entering a separate storm drain which conveys flows southerly to a combined sewer overflow conduit for eventual discharge to the Bay through outlet BOS-090. The BW&SC was notified and a maintenance crew cleaned the regulator, but only to a degree that would barely allow passage of dry weather flow to a receiving sewer. Since then, blockage has recurred, and the BW&SC maintenance crew was required to clean the regulator for a second time. It appears that the outgoing sewer, which receives the dry weather flow from the regulator, must be thoroughly cleaned in order to prevent blockage from continually reoccurring. The BW&SC is expected to perform this necessary maintenance.

Direct Dry Weather Overflows. During the regulator inspection program, two regulators that have dry weather overflow potential were discovered. The first is at the regulator located on the Dorchester interceptor near Hoyt Street (Figure C, appended). Flow through this side overflow weir-type regulator was observed to be within 2 inches of the weir crest during dry weather. It is possible that normal daily fluctuations in dry weather flow could result in the flow level exceeding the weir crest elevation, causing wastewater discharges to the CSO conduit in Auckland Street and subsequently to Dorchester Bay through outlet BOS-089. It was recommended in the 1972 report by CDM[5] that this regulator be abandoned. A similar situation exists at the regulator located in East Second Street near Farragut Road in South Boston (Figure B, appended). Dry weather flow was observed discharging over this extremely low weir and out to the Reserved Channel through outlet BOS-080. It is evident that the height of this weir should be increased.

A detailed discussion of all known dry weather discharges to Dorchester Bay is presented in Chapter V.

Sediment Buildup in Sewers. Combined sewers must adequately convey both dry weather flows and higher rates of stormwater runoff during rainfall events. Accordingly, dry weather flow velocities are typically low and are usually inadequate to maintain settleable solids in suspension, resulting in an accumulation of solids in combined sewers during dry weather periods. The settled solids are often resuspended during rainfall events, due to the high flow rates and velocities from stormwater runoff, and accumulate in downstream regulators, combined sewer overflow (CSO) conduits and interceptors. During our regulator and tide gate inspection program, several instances of sediment buildup were found, and in some cases, as mentioned previously, caused blockages which resulted in dry weather overflows to Dorchester Bay. Further documentation of sediment buildup in regulators and tide gate chambers is included in the inventory of these facilities (Appendix A).

Sediment was found in the CSO conduit that discharges to outlet BOS-090 (Figure C, appended). As previously discussed, the landfill located at this outlet represents a hydraulic restriction that reduces the flow velocity in the CSO conduit, thus enabling sedimentation to occur. Two feet of sediment was measured in this conduit while monitoring flow depth during the storm of 25 January 1979.

A large amount of sediment was found in the Dorchester interceptor during a 1977 survey by the MDC. The average sediment depth was 1.7 ft with depths of over 3 ft in some locations (the interceptor is a circular brick conduit ranging from 54 in. to 60 in. in diameter). Large amounts of sediment were also found in the siphon chambers located on the interceptor near Sturtevant Street (Figure C, appended). The average sediment depth in the 54-in. diameter portion of the interceptor between Chickatawbet Street and Freeport Street was approximately 2.5 ft. Based on a pipe slope of 0.0004 ft/ft and an assumed Manning's "n" value of 0.017, which was used throughout this study for a brick conduit, this sediment depth resulted in a reduction in flowing full carrying capacity from 25.5 mgd to 7.3 mgd or 71 percent. In the 60-in. diameter portion of the interceptor between Freeport Street and Auckland Street, the average sediment depth was about 1.4 ft. Based on the pipe slope of 0.00048 ft/ft and an "n" value of 0.017, this depth resulted in a reduction in flowing full capacity from 28.2 mgd to 22.4 mgd or 20 percent. The remaining portion of the interceptor between Auckland Street and Mt. Vernon Street had relatively minor amounts of sediment, thus the capacity was not substantially reduced.

On 26 January 1978, the MDC awarded a contract to Freaney Corporation to clean the entire length of the Dorchester interceptor from its beginning at Granite Avenue in the Neponset study area, to where it discharges to the Boston Main interceptor at Mount Vernon Street (Contract No. E-77-31S). The work was completed on 26 April 1979 with a total of approximately 2,020 cubic yards of sediment removed.

# CHAPTER V - DRY WEATHER FLOWS

## General

Information on dry weather flows in a wastewater collection system is essential to estimations of the quantity and quality of combined sewer overflows. Basically, dry weather flow is comprised of a combination of domestic sanitary sewage, industrial and commercial wastewater, and groundwater infiltration. And although the quantity of dry weather flow is relatively minor in comparison to the large volumes of stormwater required to produce an overflow, its high contaminant level has a significant impact on the quality of combined sewer overflows. The information developed on dry weather flows was utilized in several applications.

Dry weather flow information obtained during initial phases of the study was used with wet weather data for calibration of the system computer models, SEMSTORM and SWMM. Estimated average dry weather flows were used to approximate available in-system storage capacity and simulate the mixing of dry weather flow and stormwater runoff--providing estimates of overflow quality and quantity at each CSO outlet. Monitoring and sampling at the CSO outlets provided verification of the mixed flows derived from the model.

Measured dry weather flows were also required for verification of corresponding calculated values. This was considered necessary since calculated dry weather flows were based on several assumptions and estimates associated with population, water consumption, and infiltration. The methodology used to determine both measured and calculated dry weather flows and the resultant comparison of flows obtained by each method, is presented later in this chapter.

## Field Inspection, Gaging and Sampling Program

The dry weather flow field investigation program, performed from July 1978 through November 1978, consisted of field inspection/reconnaissance, and gaging/sampling. Initially, the operation and physical characteristics of the existing sewerage collection system were field inspected, in order to identify possible dry weather and wet weather gaging and sampling locations, and to locate instances of maintenance-related problems within the system, as discussed in Chapter IV.

Dry weather flow gaging and sampling was conducted at eight locations in the study area during the months of September and October 1978. Details of the program are discussed below.

Description of Gaging Equipment. Two different types of continuous recording depth-of-flow gages were used in this study: (1) a bubble-type (Bristol) gage and (2) oscillating probe-type (Manning dipper) gages. Together with additional data on sewer slope and pipe roughness characteristics, data from these depth-of-flow gages can be converted to flow measurements using Manning's formula.

The bubble-type (Bristol) gage determines flow by "sensing" the differential pressure required to force a bubble of air out of a small-diameter tube. The tube must be fastened to the wall of a manhole or pipe and extended into the flow near the pipe invert. The variation of depth is recorded on a calibrated, circular chart recorder. A 5-lb compressed air tank provides pressure for the operation of the gage and is mounted, with the recorder, on the wall of the manhole.

The oscillating probe-type gage (Manning dipper) determines depth of flow by "sensing" the water surface. This is accomplished by incorporating soil moisture and water surface into an electrical circuit completed by a wire-hung probe grounded to a manhole frame. As the probe is lowered, it makes contact with the water surface, completing the circuit, and retracts until contact is broken. This operation is repeated every four seconds and the depth of flow is plotted on a graphic recorder. This gage is particularly well suited for use in large sewers, because its installation does not require entrance into the manhole. The Bristol gage can measure depths over a range of 8 ft. The Manning Dippers have a 2-ft range, but also have an overrange tracking feature which allows the unit to record the duration of surcharge and return to normal reading without recalibration. Both types can be set to record over a 7-day or a 24-hour period.

One bubble-type (Bristol model 1G501) and five oscillating-probe type gages (Manning Dippers - model L3024) were used for the Dorchester Bay study area dry weather flow monitoring program. These gages were all obtained from the MDC. All gages were regularly inspected and maintained to minimize gage downtime.

A Gurley current meter (No. 622) was used to measure flow velocities in selected conduits during both dry and wet weather periods. The meter consists of a bucket wheel mounted on a vertical axis which revolves when suspended in flowing water. A fine wire contacts the rotating shaft and intermittently closes an electrical circuit connnected to an earphone. A small dry cell battery supplies the power. The number of clicks sounded in the earphone is counted for a time period varying from 40 to 70 seconds, and the corresponding velocity can be read directly from a rating table. The operating range of the meter is 0.2 to 23 ft/sec.

Gaging Locations. Eight locations within the Dorchester Bay study area were selected for dry weather flow gaging and sampling. These locations are described in Table V-1 and shown on Figures V-1, V-2, and V-3. Five of the gaging sites were located in major tributary areas which were considered to be representative of the entire study area. The remaining three sites were at key locations on the South Boston and Dorchester interceptors.

The locations were selected based on the following criteria:

1. Accessibility provided by a manhole located directly over the trunk sewer rather than "offset."

2. A constant slope and cross section as the sewer approached, entered and left the gaging manhole.

3. Lack of appreciable sediment buildup in the gaging manhole and the conduit containing the waste stream.

4. Absence of backwater effects or hydraulic restriction that could affect the flow characteristics at the gaging site.

5. Structural conditions and location (with regard to traffic conditions, accessibility, etc.) suitable for equipment installation.

Several desirable gaging locations were rejected because they failed to meet the above criteria on field inspection.

The choice of gaging sites located on the interceptors was based on the necessity to obtain data on the total dry weather flows for the study area. Flow monitoring was required at: (1) the upper end of the Dorchester Interceptor (at McKone Street) where flows enter the Dorchester Bay study area from the Neponset study area; (2) the lower end of the Dorchester Interceptor (near Morrissey Boulevard) where it joins the Boston Main Interceptor; and (3) the lower end of the South Boston Interceptor (near Mt. Vernon Street) where it joins the Boston Main Interceptor (see Figures V-1, V-2, and V-3). Field investigations determined the best manhole for gaging at each of these locations.

In order to quantify dry weather flows emanating from the South Boston portion of the study area, a gaging site in addition to (3) above, was desired on the south branch of the South Boston Interceptor upstream of its junction with the north branch at Devine Way. However, backwater conditions encountered at this junction, which extended upstream for a considerable distance, would have prevented collection of reliable flow data. Flows gaged further upstream of this condition would not have been representative of the dry weather flow from the entire South Boston tributary area.

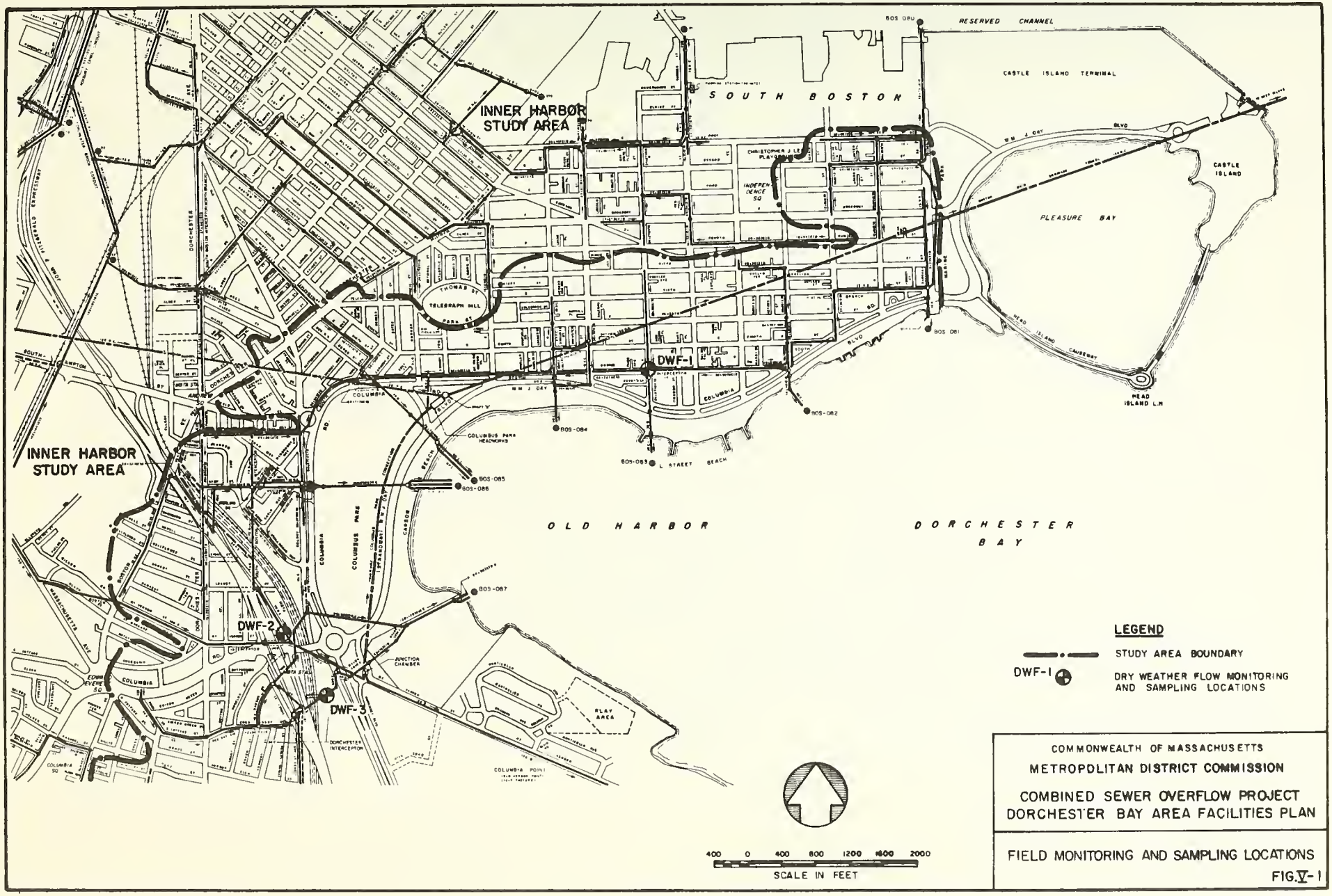
TABLE V-1

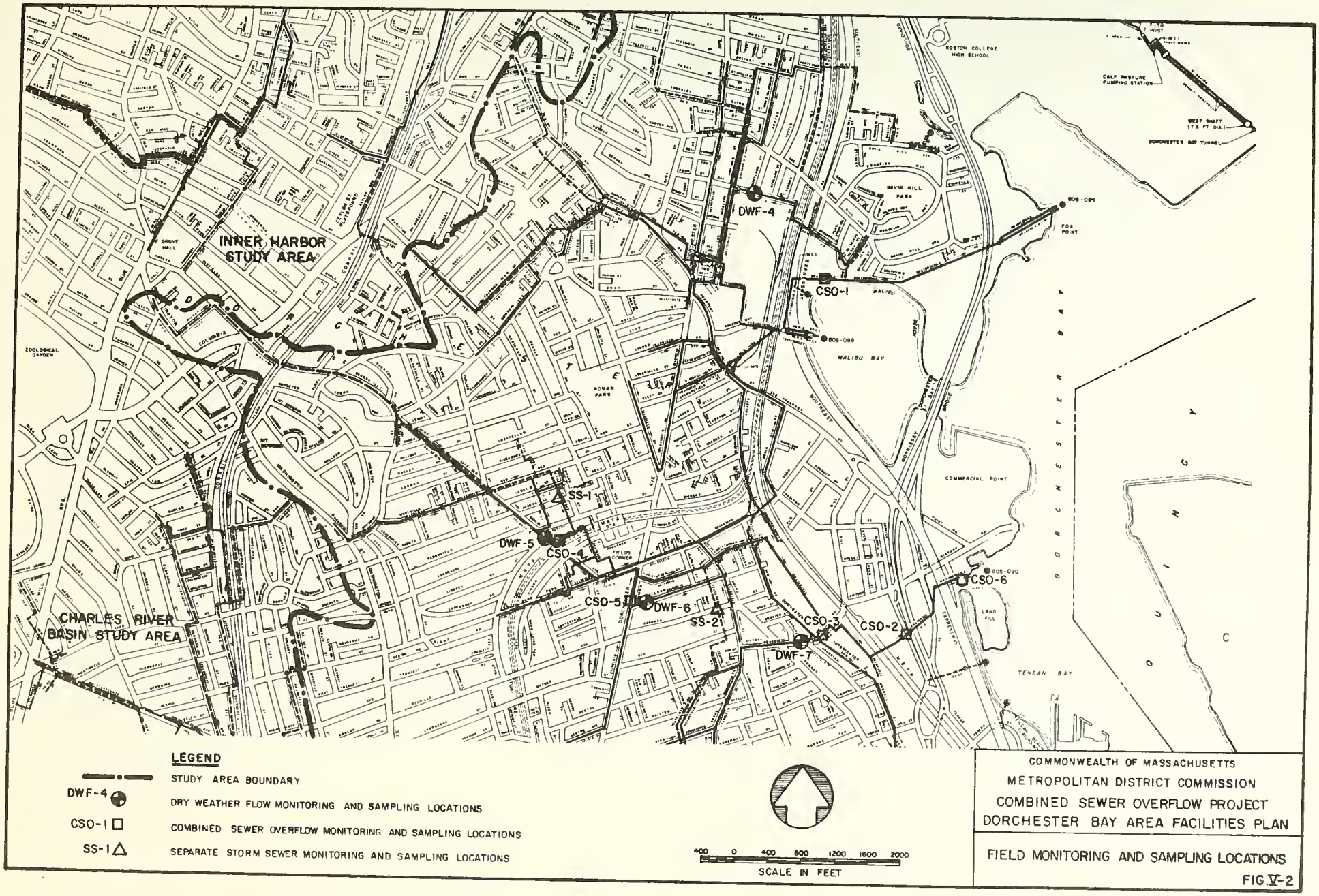
SUMMARY OF THE DRY WEATHER FLOW GAGING AND SAMPLING PROGRAM
FOR THE DORCHESTER BAY STUDY AREA

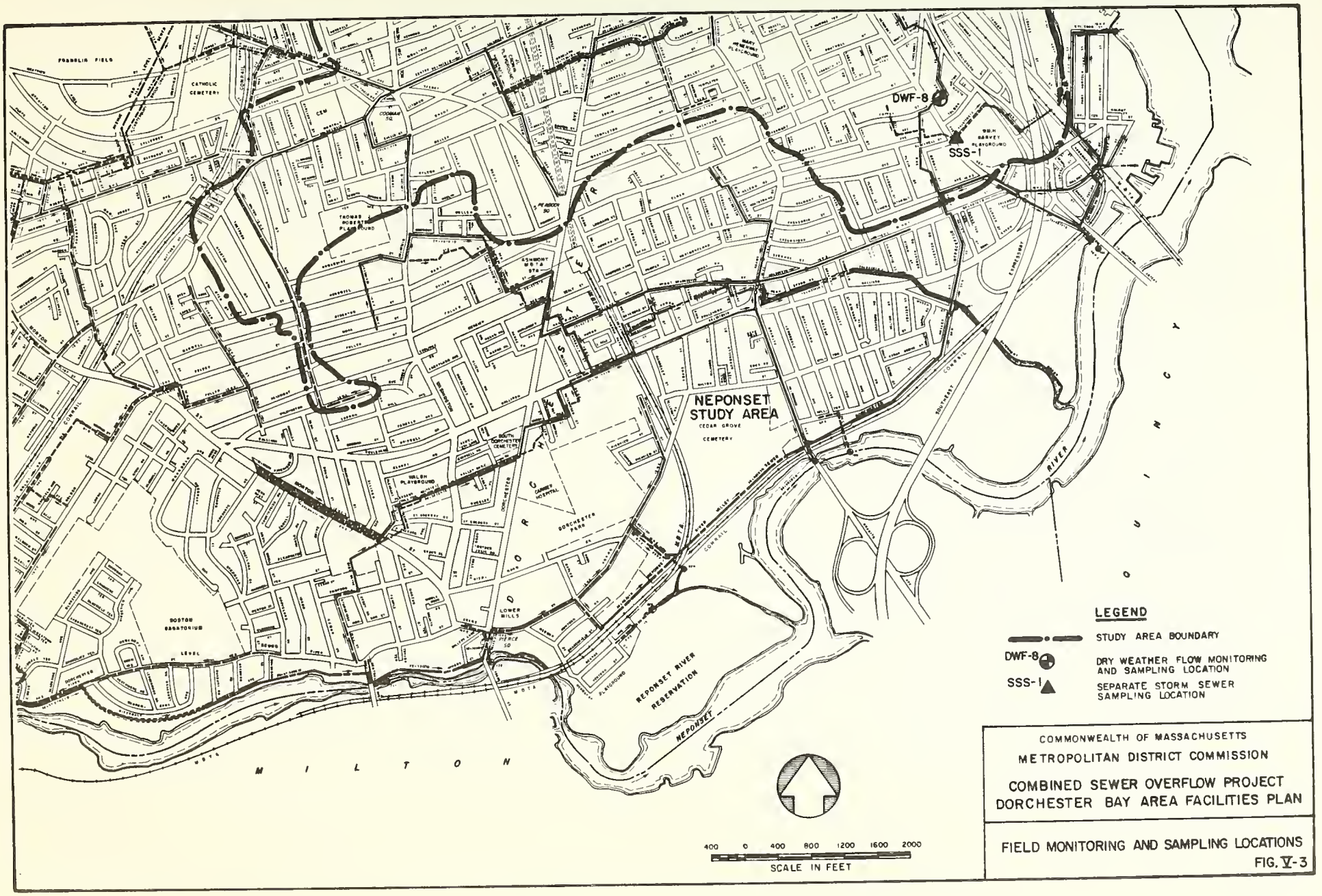
| LOCATION NO.* | DESCRIPTION | GAGE TYPE | MONITORING PERIOD** (1978) | SAMPLING DATE | COMMENTS |
|---|---|---|---|---|---|
| DWF-1 | K St. at Marine Rd. | Manning Dipper | 9/8-9/13 | 9/7 | |
| DWF-2 | South Boston Interceptor at Mt. Vernon St. | Manning Dipper | 10/6-10/12 | 10/12 | Total flow in interceptor includes north (Inner Harbor) and south branches. |
| DWF-3 | Dorchester Interceptor at Morrissey Blvd. | Manning Dipper | 10/6-10/12 | 10/12 | Total flow in interceptor includes that from Neponset area. |
| DWF-4 | Bay St. at Maryland St. | Manning Dipper | 9/19-9/25 | 9/21 | |
| DWF-5 | Geneva Ave. at Tonawanda St. | Bristol Gage | 9/15-9/19 | 9/14 | |
| DWF-6 | Gibson St. near Dorchester Ave. | Manning Dipper | 10/10-10/12 | 9/28 | |
| DWF-7 | Victory Rd. at Neponset St. | Manning Dipper | 9/25-10/2 | 9/28 | |
| DWF-8 | Dorchester Interceptor at McKone St. | Manning Dipper | 10/18-10/25 | none at this location | Monitoring required to determine flow entering Dorchester Bay study area from Neponset study area. |

*Locations are shown on Figures V-1, V-2 and V-3.

**Data obtained between the dates indicated best represent typical dry weather flow conditions. These dates do not reflect the total gage installation period.


INNER HARBOR STUDY AREA
INNER HARBOR STUDY AREA
SOUTH BOSTON
RESERVED CHANNEL
CASTLE ISLAND TERMINAL
CASTLE ISLAND
PLEASURE BAY
MEAD ISLAND L.H.
TELEGRAPH HILL
DWF-1
DWF-2
DWF-3
BOS-080
BOS-081
BOS-082
BOS-083
BOS-084
BOS-085
BOS-086
BOS-087
OLD HARBOR
DORCHESTER BAY
L STREET BEACH
COLUMBUS PARK HEADWORKS
JUNCTION CHAMBER
DORCHESTER INTERCEPTOR
FLAT AREA
COLUMBIA POINT
LEGEND
STUDY AREA BOUNDARY
DRY WEATHER FLOW MONITORING AND SAMPLING LOCATIONS
400 0 400 800 1200 1600 2000
SCALE IN FEET
COMMONWEALTH OF MASSACHUSETTS
METROPOLITAN DISTRICT COMMISSION
COMBINED SEWER OVERFLOW PROJECT
DORCHESTER BAY AREA FACILITIES PLAN
FIELD MONITORING AND SAMPLING LOCATIONS
FIG. V-1


INNER HARBOR STUDY AREA
CHARLES RIVER BASIN STUDY AREA
DWF-4
DWF-5
DWF-6
DWF-7
CSO-1
CSO-2
CSO-3
CSO-4
CSO-5
CSO-6
SS-1
SS-2
MALIBU BAY
DORCHESTER BAY
COMMERCIAL POINT
TENEAN BAY
FOX POINT
CALF PASTURE PUMPING STATION
BOSTON COLLEGE HIGH SCHOOL
ZOOLOGICAL GARDEN
LEGEND
STUDY AREA BOUNDARY
DRY WEATHER FLOW MONITORING AND SAMPLING LOCATIONS
COMBINED SEWER OVERFLOW MONITORING AND SAMPLING LOCATIONS
SEPARATE STORM SEWER MONITORING AND SAMPLING LOCATIONS
400 0 400 800 1200 1600 2000
SCALE IN FEET
COMMONWEALTH OF MASSACHUSETTS
METROPOLITAN DISTRICT COMMISSION
COMBINED SEWER OVERFLOW PROJECT
DORCHESTER BAY AREA FACILITIES PLAN
FIELD MONITORING AND SAMPLING LOCATIONS
FIG. V-2


DWF-8
SSS-1
NEPONSET
STUDY AREA
CEDAR GROVE
CEMETERY
DORCHESTER BAY
NEPONSET RIVER
RESERVATION
NEPONSET
RIVER
M I L T O N
LEGEND
STUDY AREA BOUNDARY
DWF-8
DRY WEATHER FLOW MONITORING AND SAMPLING LOCATION
SSS-1
SEPARATE STORM SEWER SAMPLING LOCATION
400 0 400 800 1200 1600 2000
SCALE IN FEET
COMMONWEALTH OF MASSACHUSETTS
METROPOLITAN DISTRICT COMMISSION
COMBINED SEWER OVERFLOW PROJECT
DORCHESTER BAY AREA FACILITIES PLAN
FIELD MONITORING AND SAMPLING LOCATIONS
FIG. V-3

Gaging and Sampling Procedures. At each gaging location, depth-of-flow measurements were continuously recorded for about seven days during dry weather conditions. Within one 24-hour period during that week, grab samples (2,000 ml each) were taken every three hours (a total of eight samples at each location). The gages were equipped with 24-hour recording charts during the sampling period; otherwise, charts calibrated to record for seven days were used. Samples were taken in sterilized plastic bottles and placed in an iced cooler for transport to the CDM laboratory for analysis.

In addition to the eight continuous gaging locations delineated above, spot night gaging was performed throughout the study area to obtain information on system infiltration rates. Usually, a major portion of the dry weather flows gaged during the early morning hours is infiltration, since, in most cases, the contributions from domestic, commercial, and industrial sources are minimal at this time. To ensure this was the case, gaging locations were carefully selected to minimize any possible influence of continuous industrial and/or commercial wastewater discharges.

Night gaging was conducted between the hours of 1:00 and 3:00 AM on four separate nights during the month of September. Specific nights were chosen to coincide with different tidal cycles, since tidal influence has major effects on infiltration in some areas (e.g., the South Boston Interceptor) as will be discussed later in this chapter. Depth-of-flow measurements were taken at each of the dry weather flow locations previously mentioned (with the exception of the Dorchester interceptor at McKone Street), in addition to several other locations. Depth of flow was measured by use of a calibrated surveyor's rod coated with a hydrous indicator. This information, together with physical characteristics of the sewer determined by field investigation, was used to estimate infiltration rates for the tributary area. The results of the night gaging program will be discussed later in this chapter.

If pipe dimensions and slope are known, simultaneous measurement of velocity and depth of flow allows one to calculate to pipe roughness coefficient (Mannings "n" value). For the purposes of refining flow data in this manner, velocity and depth-of-flow measurements were taken at each of the dry weather flow locations and the roughness coefficient for each conduit being monitored was estimated.

Generally, sampling was performed in accordance with the 16 August 1978 "Quality Assurance Plan/Monitoring and Sampling Guidelines" proposed by Camp Dresser & McKee Inc. for the Dorchester Bay study area. Grab samples were analyzed for BOD , TSS, total and fecal coliform, and five heavy metals, (Cadmium, Chromium, Lead, Mercury, Zinc). Dissolved oxygen (DO) and pH were measured in the field.

## Existing Dry Weather Flow Quantity

General. Existing dry weather flows from the Dorchester Bay study area were calculated using estimates of present population, water consumption and infiltration. As discussed in Chapter III, existing dry weather wastewater flows also represent design year (2005) flows since no significant increases in population and water consumption are expected. Since certain estimates or assumptions were used in this quantification procedure, it was necessary to determine the validity of these estimates by comparing them to field measured dry weather flows obtained during the dry weather flow monitoring and sampling program. Results of this comparison generally indicated compatibility of calculated dry weather flows and field measured values.

Thus, calculated dry weather flow estimates were used in the preparation of this facilities plan. The methodology used to calculate dry weather flows, as well as a discussion of the field measured results, is presented in the following sections of this chapter.

Dry Weather Flow Quantification Procedure. Figure V-4 summarizes the methodology used to estimate the current average dry weather flow for each sewered subarea within the Dorchester Bay study area. Total metered water consumption for Wards 13 and 15, which are considered representative of the study area and are generally located within the central portion of the study area, was obtained for the year 1977 from Boston Water and Sewer Commission (BW&SC) water meter records.

Major industrial and institutional users were identified, and their 1977 metered water use was determined. For the purposes of this study, any user with a water consumption volume greater than 100,000 cubic feet (748,000 gallons) in the year 1977 was defined as a major user. These include the University of Massachusetts, Boston College High School, the Boston Globe, the First National Bank of Boston office building, and several laundries, car washes, small industries and commercial establishments.

All metered consumption values were then increased by 20 percent to account for meter slippage and unmetered usage (unaccounted-for water). This value was obtained from discussions with the Boston Water and Sewer Commission, and from the March 1978 "Deer Island Infiltration/Inflow Study" by CDM, [8] in which investigations showed that the unaccounted-for water in several study area communities ranged from 15 to 25 percent. An average of 20 percent was used in that report, and was also used in the November 1975 "EMMA Study" by Metcalf and Eddy [9], in cases where unaccounted-for water was not known. From the adjusted ward totals, the adjusted major industrial and institutional consumption was subtracted. The remaining adjusted consumption was then divided by the current (1975) estimated ward population, to obtain domestic and minor nondomestic (i.e., small business' and industries) per-capita water consumptions of 68 gal./day/cap. for Ward 15 and 77 gal./day/cap. for Ward 13. For the purpose of this study, an average value of 72 gal/day/cap was used for each sewered subarea within the Dorchester Bay study area.

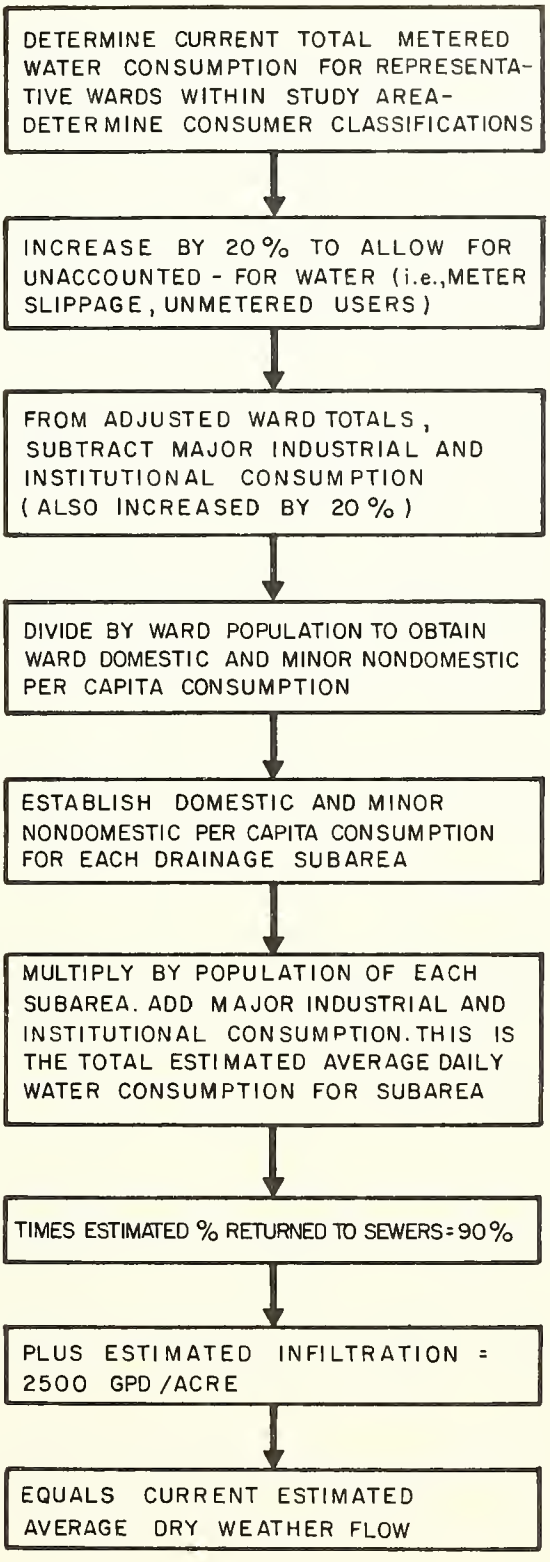

DETERMINE CURRENT TOTAL METERED WATER CONSUMPTION FOR REPRESENTATIVE WARDS WITHIN STUDY AREA- DETERMINE CONSUMER CLASSIFICATIONS
INCREASE BY 20% TO ALLOW FOR UNACCOUNTED - FOR WATER (i.e.,METER SLIPPAGE, UNMETERED USERS)
FROM ADJUSTED WARD TOTALS, SUBTRACT MAJOR INDUSTRIAL AND INSTITUTIONAL CONSUMPTION (ALSO INCREASED BY 20%)
DIVIDE BY WARD POPULATION TO OBTAIN WARD DOMESTIC AND MINOR NONDOMESTIC PER CAPITA CONSUMPTION
ESTABLISH DOMESTIC AND MINOR NONDOMESTIC PER CAPITA CONSUMPTION FOR EACH DRAINAGE SUBAREA
MULTIPLY BY POPULATION OF EACH SUBAREA. ADD MAJOR INDUSTRIAL AND INSTITUTIONAL CONSUMPTION. THIS IS THE TOTAL ESTIMATED AVERAGE DAILY WATER CONSUMPTION FOR SUBAREA
TIMES ESTIMATED % RETURNED TO SEWERS=90%
PLUS ESTIMATED INFILTRATION = 2500 GPD/ACRE
EQUALS CURRENT ESTIMATED AVERAGE DRY WEATHER FLOW

FIG. V-4 DRY WEATHER FLOW QUANTIFICATION PROCEDURE

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METROPOLITAN DISTRICT COMMISSION
COMBINED SEWER OVERFLOW PROJECT
DORCHESTER BAY AREA FACILITIES PLAN

Accordingly, the total estimated average daily water consumption for each subarea was determined by multiplying 72 gal./day/cap. by the estimated subarea population, and adding to this the subarea's major industrial and institutional consumption. To compute the estimated average wastewater flow for each subarea, a return ratio of 0.90 was used. This ratio, which represents the estimated proportion of water use discharged or returned to the sewer, appears to be reasonable for this study area where lawn watering and car washing probably occur somewhat less frequently than in suburban areas. This return ratio was also used in the EMMA Study [9] for areas developed to a density of 30 persons per acre or greater (the Dorchester Bay study area has a density of about 38 persons per sewered acre). The current estimated average dry weather flow for each sewered subarea was then determined by adding an allowance for groundwater infiltration of 2,500 gallons per day per acre (gpd/acre). This rate was based on the results of the dry weather flow monitoring program and assumes that 80 percent of the minimum measured dry weather flow was comprised of groundwater infiltration. This rate also agrees with the results of the March 1978 infiltration/inflow (I/I) study by CDM [8], in which 2,500 gpd/acre was the annual average infiltration rate for the areas tributary to the Dorchester and South Boston interceptors.

As indicated earlier, night gaging was also performed to obtain further information on infiltration rates within the study area. As will be discussed later in this chapter, the results were generally not compatible with those obtained from the dry weather flow monitoring program and, consequently, were not used in this study.

Results of the dry weather flow quantification procedure performed for each sewered subarea are given in Table V-2. As shown, the total estimated average dry weather flow for the Dorchester Bay study area is 12.67 mgd. This results in an overall per capita discharge (including major industrial and institutional discharges and infiltration) of 137 gal./day/cap. Table V-2 also indicates the locations of dry weather flow inputs to the study area's interceptor system, with the relative location of each input shown schematically on Figure V-5. As indicated in Table V-2, most of the dry weather flow inputs are from regulator dry weather flow connections, except for three areas generally located east of the Dorchester interceptor and served by separate storm and sanitary sewers (Figures B, C, and D, appended). The dry weather sanitary flow from these areas enters the Dorchester interceptor through several small direct connections; however, for the purposes of simplifying Figure V-5, the total estimated dry weather flow from each of these three areas is shown entering the Dorchester interceptor at a single location per area.

Flows shown on Figure V-5 that enter the Dorchester Bay study area from the Inner Harbor and Neponset study areas will be discussed later in this chapter.

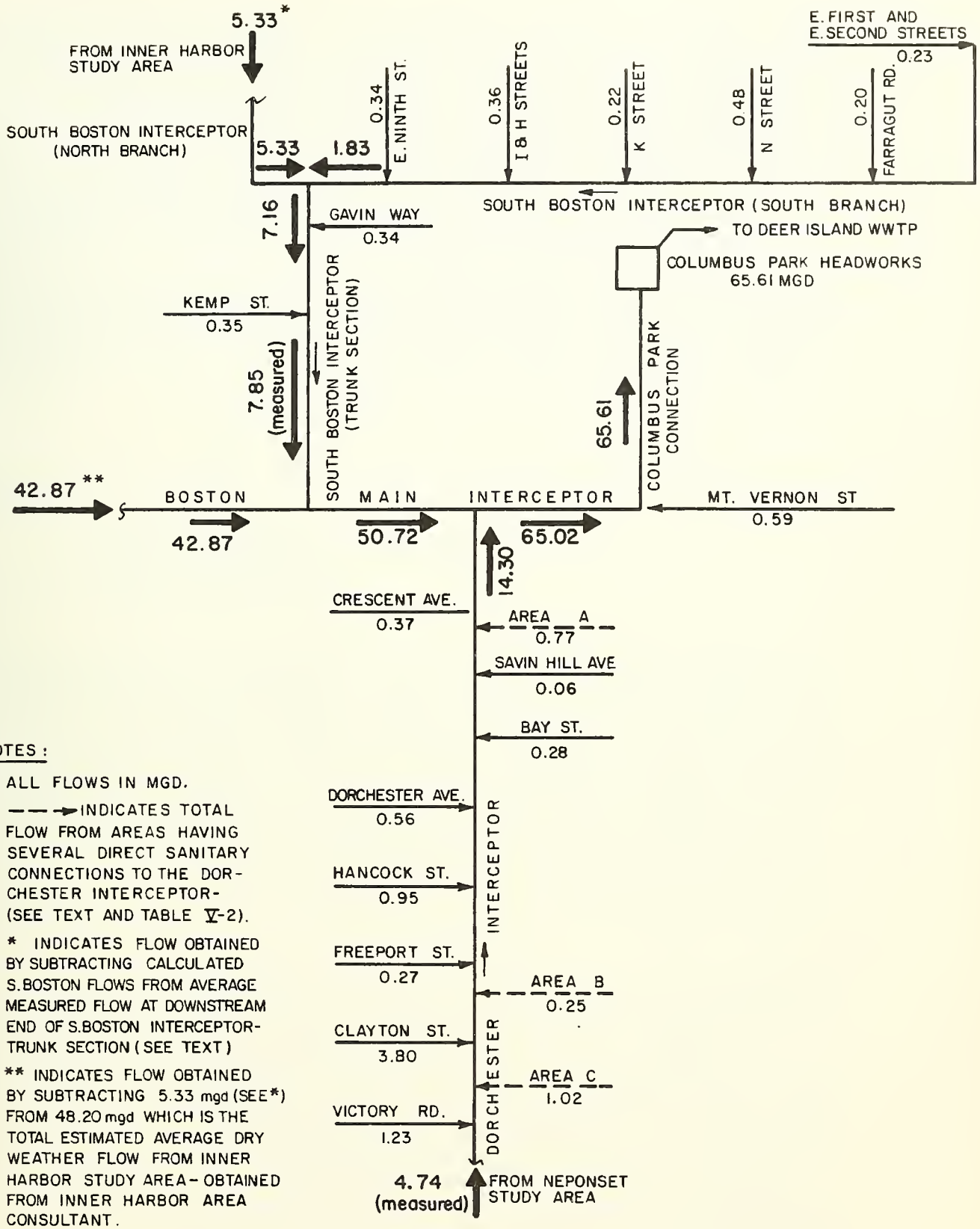
TABLE V-2. ESTIMATED EXISTING AVERAGE DAILY DRY WEATHER FLOWS FOR THE DORCHESTER BAY STUDY AREA

| Location of Dry Weather Flow Input To Interceptor System[1] | Tributary Area, Acres | Estimated Tributary Population | ESTIMATED AVERAGE DAILY DRY WEATHER FLOWS, MGD | | | |
|---|---|---|---|---|---|---|
| | | | Domestic + Minor Non-Domestic[2] | Major Industrial and Institutional | Infiltration[3] | Total Average Dry Weather Flow |
| **So. Boston** | | | | | | |
| E. First & E. Second Sts. | 47 | 1713 | 0.11 | - | 0.12 | 0.23 |
| Farragut Rd. | 34 | 1745 | 0.11 | - | 0.09 | 0.20 |
| N. St. | 72 | 4663 | 0.30 | - | 0.18 | 0.48 |
| K. St. | 29 | 2242 | 0.15 | - | 0.07 | 0.22 |
| I&H Sts. | 53 | 3517 | 0.23 | - | 0.13 | 0.36 |
| E. Ninth St. | 47 | 3351 | 0.22 | - | 0.12 | 0.34 |
| Kemp St. | 55 | 2288 | 0.15 | 0.06 | 0.14 | 0.35 |
| Gavin Way | 51 | 3307 | 0.21 | - | 0.13 | 0.34 |
| **Dorchester** | | | | | | |
| Crescent Ave. | 68 | 2931 | 0.19 | 0.01 | 0.17 | 0.37 |
| Mt. Vernon St. | 55 | 4190 | 0.27 | 0.18 | 0.14 | 0.59 |
| Area A(4) | 141 | 2398 | 0.16 | 0.26 | 0.35 | 0.77 |
| Savin Hill Ave. | 9 | 551 | 0.04 | - | 0.02 | 0.06 |
| Bay St. | 65 | 1525 | 0.10 | 0.02 | 0.16 | 0.28 |
| Dorchester Ave. | 113 | 4185 | 0.27 | 0.01 | 0.28 | 0.56 |
| Area B(4) | 86 | 349 | 0.02 | 0.01 | 0.22 | 0.25 |
| Area C(4) | 271 | 5256 | 0.34 | - | 0.68 | 1.02 |
| Hancock St. | 167 | 8128 | 0.53 | - | 0.42 | 0.95 |
| Freeport St. | 55 | 1829 | 0.12 | 0.01 | 0.14 | 0.27 |
| Clayton St. | 782 | 28,308 | 1.83 | 0.01 | 1.96 | 3.80 |
| Victory Rd. | 227 | 10,152 | 0.66 | - | 0.57 | 1.23 |
| TOTALS | 2,427 | 92,628 | 6.01 | 0.57 | 6.09 | 12.67 |

NOTES:

(1) All locations are shown on Fig. V-5 and are at regulators unless otherwise indicated.
(2) 72 gal/day/cap x tributary population x 0.9 (return ratio).
(3) 2500 gal/day/acre x tributary area.
(4) Areas served by several small direct sanitary connections to the Dorchester Interceptor located as follows (refer to Figures B, C, and D appended).

Area A - east of the Dorchester Interceptor between Crescent Ave. extended and Savin Hill Ave.
Area B - east of the Dorchester Interceptor between Bay St. and Park St.
Area C - separated areas located between Christopher Street and the southern boundary of the study area, generally east of the Dorchester Interceptor.




NOTES :

1. ALL FLOWS IN MGD.
2. ——→ INDICATES TOTAL FLOW FROM AREAS HAVING SEVERAL DIRECT SANITARY CONNECTIONS TO THE DORCHESTER INTERCEPTOR- (SEE TEXT AND TABLE V-2).
3. * INDICATES FLOW OBTAINED BY SUBTRACTING CALCULATED S.BOSTON FLOWS FROM AVERAGE MEASURED FLOW AT DOWNSTREAM END OF S.BOSTON INTERCEPTOR-TRUNK SECTION (SEE TEXT)

   ** INDICATES FLOW OBTAINED BY SUBTRACTING 5.33 mgd (SEE*) FROM 48.20 mgd WHICH IS THE TOTAL ESTIMATED AVERAGE DRY WEATHER FLOW FROM INNER HARBOR STUDY AREA - OBTAINED FROM INNER HARBOR AREA CONSULTANT.

FIG. V-5 SCHEMATIC OF ESTIMATED AVERAGE DRY WEATHER FLOW INPUTS TO THE DORCHESTER BAY STUDY AREA INTERCEPTOR SYSTEM

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METROPOLITAN DISTRICT COMMISSION
COMBINED SEWER OVERFLOW PROJECT
DORCHESTER BAY AREA FACILITIES PLAN

Measured Dry Weather Flows. As mentioned previously, dry weather flows were measured at eight locations within the Dorchester Bay study area. The locations are shown on Figures V-1, V-2, and V-3 and listed on Table V-3. Table V-3 also contains the results of the field measurements including maximum, minimum and average dry weather flows and their tributary area and population. Calculated dry weather flows, determined by the method previously discussed, are also shown for comparison. A discussion of this comparison is presented later in this chapter. The methodology of estimating dry weather flows from field measurements is discussed in the following portion of this chapter.

In order to facilitate flow calculation from depth-of-flow measurements, a computer program was employed which uses Manning's equation to compute flow rates based on water level readings. Manning's equation is as follows:

$$Q = \frac{1.49}{n} R^{2/3} So^{1/2} A$$

where:

Q = flow (cfs)
n = Manning's roughness coefficient
R = hydraulic radius (ft)
So = pipe slope (ft/ft)
A = cross-sectional area of flow (ft )

The above equation holds true for any depth of flow within a pipe, if necessary hydraulic conditions are met. The above equation assumes that the pipe is long enough so that "normal" depth of flow is attained. Simply expressed, normal depth is attained when the water surface slope along the pipe is equal to the pipe slope. The above equation also assumes that the parameters (Q, So, n, and pipe diameter) are constant throughout the length of the pipe, and that the pipe alignment is straight. A small variation in flow, because of infiltration or inflow, would not seriously affect the computations. Depth metering station locations were selected to adequately measure flow from a particular tributary area, as well as to reasonably satisfy the above hydraulic conditions.

Flows are normally calculated using assumed Manning "n" values similar to those which were presented in Chapter III (i.e., 0.015 for concrete and vitrified clay conduits, and 0.017 for conduits of brick, wood, stone, clay, plain concrete, iron and undetermined materials). However, research [10] indicates that significant errors in flow estimates can occur, due to inaccurate "n" value estimates. Since depth of flow, sediment deposits, and structural condition of the sewer affects the roughness coefficient, velocity measurements were required to estimate actual "n" values in the manner discussed previously. Thus, at all dry weather flow monitoring locations, measured "n" values were used to calculate flows. Velocity readings were taken in mid-channel and at a depth of two-thirds of the distance from the water surface, where average velocity

TABLE V-3. RESULTS OF DRY WEATHER FLOW MONITORING PROGRAM
FOR THE DORCHESTER BAY STUDY AREA

| Monitor Designation(1) | Monitoring Location(1) | Tributary Area, acres | Estimated Tributary Population | Measured Daily Dry Weather Flows, mgd(6) | | | Max: Ave. DWF Ratio | Calculated Average Dry Weather Flows - mgd(2) | Meas. Ave. DWF / Calc. Ave. DWF |
|---|---|---|---|---|---|---|---|---|---|
| | | | | Min. | Ave. | Max. | | | |
| DWF-1 | K. St. @ Marine Rd. | 29 | 2,242 | 0.03 | 0.11 | 0.19 | 1.7:1 | 0.22 | 0.5 |
| DWF-2 | S. Boston Interceptor near Mt. Vernon St. | 962(4) | 31,826(4) | 4.56 | 7.85 | 12.19 | 1.5:1 | - | - |
| DWF-3 | Dorchester Interceptor near Morrissey Blvd. | 2759 | 85,412 | 11.47 | 14.90 | 17.63 | 1.2:1 | 14.30(3) | 1.04 |
| DWF-4 | Bay St. @ Maryland St. | 65 | 1,525 | 0.36 | 0.55 | 0.95 | 2:1 | 0.28 | 1.96 |
| DWF-5 | Geneva Ave. @ Tonawanda St. | 278 | 12,582 | 0.84 | 1.07 | 1.64 | 1.5:1 | 1.51 | 0.71 |
| DWF-6 | Gibson St. @ Dorchester Ave. | 251 | 8,104 | 0.92 | 1.17 | 1.44 | 1.2:1 | 1.15 | 1.02 |
| DWF-7 | Victory Rd. @ Neponset Ave. | 227 | 10,152 | 3.40 | 4.09 | 4.76 | 1.1:1 | 1.23 | 3.33 |
| DWF-8 | Dorchester Interceptor @ McKone St. | 800(5) | 19,800(5) | 1.34 | 4.74 | 6.83 | 1.4:1 | - | - |

Notes:

(1) Monitoring locations are shown on Figures V-1, V-2 and V-3.

(2) Based on the flow quantification procedure shown on Figure V-4.

(3) Includes 4.74 mgd from the Neponset study area which was field measured by CDM.

(4) Includes data obtained from the Inner Harbor study area consultant (O'Brien & Gere, Engineers, Inc.)

(5) Obtained from Neponset study area consultant (Havens & Emerson, Inc.)

(6) Excludes flows resulting from wet weather events that may have occurred during dry weather flow monitoring periods.

might be expected. Several readings were made during each determination, to ensure a constant result. The depth of flow was measured by a surveyor's rod treated with a hydrous indicator. Two such velocity measurement programs were conducted, to determine the consistency of results at random depths and times of day.

The slope of the conduit located upstream of each dry weather flow monitoring location was also determined in the field, by utilizing a surveyor's transit and level rod to measure the relative difference in pipe invert elevation. The measured slopes were then compared to slopes obtained from the Boston Water and Sewer Commission sewerage system plats, to assess the accuracy of the plat information, which was utilized in the development of the network model. Significant discrepancies were found at the Victory Road (DWF-7) and Geneva Avenue (DWF-5) locations where the measured slopes were considerably less than the slopes obtained from the plats. Minor discrepancies existed at the other monitoring locations. In all cases, field measured slopes were used for flow calculations.

In addition to Table V-3, results of the dry weather flow monitoring program are presented in Appendix A. For each dry weather flow monitoring location, a computer printout is included which displays the field monitoring results and includes average, maximum and minimum flow rates for each day during the gaging period. A hydrograph was also developed for each location, which graphically displays the hourly flow fluctuations (see Appendix A). Rainfall occurred during dry weather monitoring at several locations, and is shown in the form of a hyetograph on the dry weather flow hydrograph figures.

In general, the hydrographs show a diurnal flow fluctuation at most of the monitoring locations, some more pronounced than others. However, abnormalities exist at DWF-2 (South Boston interceptor near Mt. Vernon Street) and DWF-4 (Bay Street near Maryland Street). At DWF-2, where the trunk section of the South Boston interceptor was gaged near its point of discharge to the Boston main interceptor (Figure V-1), the flow rate appears to be tidally influenced. This is based on an examination of the tidal cycles occurring during the period of gaging. Referring to the DWF-2 hydrograph, the peaks that occur in the vicinity of 6:00 AM on each day of gaging generally coincide with the times of high tide. The north branch of the South Boston interceptor is probably the major source of the tide water. Examination of the Inner Harbor consultant's gaging results on the north branch show a similar, tidally influenced flow pattern with peaks occurring coincident with high tides.

At DWF-4, the normal, expected diurnal flow pattern was not evident. Field investigations revealed that the Boston Insulated Wire and Cable Company, located on Bay Street, periodically discharges cooling water to the Bay Street combined sewer upstream of the gaging location. The company, which operates 24 hours per day, discharges cooling water to a storage pond. From there it is either recirculated back through the

plant or pumped directly to the sewer. Pumping to the sewer is controlled by the water level in the pond, and does not occur at regular intervals. According to a company official, the cooling water contains no process wastes or contaminants.

As previously mentioned, night gaging was also performed in order to estimate infiltration rates within the Dorchester Bay study area. A total of 12 locations were monitored on three separate occasions, in accordance with the procedures discussed previously in this chapter. In general, results varied and exhibited no definitive pattern. Only four of a total of twelve night gaging locations showed infiltration rates comparable to those computed from dry weather flow monitoring program results, and to those presented in CDM's March 1978 infiltration/inflow (I/I) study [8] for areas tributary to the Dorchester and South Boston interceptors (2,500 gal./day/acre). The four locations were as follows:

| | |
|---|---|
| Geneva Avenue at Tonawanda Street - | 2,300 gal./day/acre |
| Bay Street at Maryland Street - | 2,800 gal./day/acre |
| Freeport Street at Clayton Street - | 2,800 gal./day/acre |
| Norfolk Street at Oakwood Street - | 3,000 gal./day/acre |

Infiltration rates at the remaining eight locations ranged from 4,500 gal./day/acre to 48,000 gal./day/acre. At locations near the shoreline, no tidal pattern was evident, and, in some cases, results indicated lower flows during high tides than during low tides.

One reason for these high rates is probably inaccurate depth-of-flow measurements taken at the manholes used for gaging, possibly due to sediment deposits and/or structural irregularities of the manhole bottom. Also, measuring the depth of flow of a high velocity waste stream by using a surveyor's level rod could cause inaccurate depth of flow readings when using a hydrous indicator, due to the rise of water on the upstream side of the rod.

Accordingly, it was considered more accurate to use results obtained from the dry weather flow monitoring program, since they, for the most part, compared favorably with calculated values on an average flow basis (Table V-3). Referring to Table V-3, the infiltration rate was estimated by averaging the minimum measured dry weather flow rates on a gallons per day per acre (gal./day/ acre) basis, and assuming that 80 percent of this figure represented infiltration.* This resulted in an average study area infiltration rate of about 2,500 gal./day/acre. The results of CDM's March 1978 I/I study[8] which indicate an infiltration rate of 2,500 gal./day/acre for areas tributary to the Dorchester and South Boston interceptors, further justifies the use of this rate for the Dorchester Bay study area.

*Measured minimum flows at DWF-7 were not used because the large difference between calculated and measured values are most likely due to field gaging inaccuracies (Refer to Table V-3).

The above analysis was necessary to establish dry weather flow estimates for the above area. However, it should be emphasized that infiltration in combined sewers is a small percentage of the total volumes that are required to generate combined sewer overflows, thus have a minimal effect on overflow volume and frequency.

Comparison of Calculated and Measured Dry Weather Flows. As previously mentioned, Table V-3 compares calculated and measured dry weather flows. Comparisons were made at six dry weather flow monitoring locations, with the results shown in the form of a ratio of measured vs. calculated values. The closer the ratio is to 1.0, the closer the two values. As shown on Table V-3, calculated average dry weather flows compared favorably to corresponding measured values at DWF-3 (ratio = 1.04) and DWF-6 (Ratio = 1.02). A reasonably close parity was also obtained at DWF-5 (Ratio = 0.71). Field gaging inaccuracies were most likely the cause of the discrepancies at monitoring locations DWF-1 (Ratio = 0.50), DWF-4 (Ratio - 1.96), and DWF-7 (Ratio = 3.33). However, the fact that measured values were within 4.2 percent of calculated values at the Dorchester interceptor near Morrissey Boulevard (DWF-3), indicates that the assumptions used to calculate dry weather flows were reasonable and generally representative of the study area, since about 83 percent of the Dorchester Bay study area and the entire Neponset study area is tributary to this location.

Average Dry Weather Flow Balance. Also shown on Figure V-5 is the magnitude and direction of all estimated dry weather flows entering and exiting (i.e., Columbus Park Headworks) the Dorchester Bay study area. The flows shown were derived from three source of data: (1) estimated, (2) CDM field gaging, and (3) adjoining study areas. Estimated average dry weather flows entering the Dorchester Bay study area from the Inner Harbor study area are conveyed through the Boston Main Interceptor, and the North Branch of the South Boston interceptor; from the Neponset study area, through the southern portion of the Dorchester interceptor.

The estimated current average dry weather total flow entering the Dorchester Bay study area from the Inner Harbor study area is 48.20 mgd. As part of CDM's field program, the downstream end of the trunk section of the South Boston interceptor was gaged. Flow tributary to this gaging location includes both the north and south branches of the South Boston interceptor. The average measured dry weather flow rate was 7.85 mgd. Subtracting the calculated dry weather flow for the South Boston portion of the study area from the 7.85 mgd gaged flow rate, results in a CDM proportioned dry weather flow rate of 5.33 mgd from the north branch of the South Boston interceptor (Inner Harbor). Accordingly, for use in Fig. V-5, the following Inner Harbor flow allocation was used:

42.87 mgd - Boston Main Interceptor
5.33 mgd - North Branch of South Boston Interceptor
48.20 mgd - Total Entering Dorchester Bay Study Area

This flow allocation was discussed with the Inner Harbor area consultant, and it was agreed that this method of flow proportioning most likely best represents actual field conditions, especially when considering the relative differences in the tributary area of each interceptor (i.e., 10,220 acres for Boston Main interceptor, and 574 acres for the north branch of the South Boston interceptor).

Flows from the Neponset study area were based on CDM field monitoring results at the gaging station located on the Dorchester interceptor at McKone Street, near the study area boundary (DWF-8). The average dry weather flow during the one week gaging period was 4.74 mgd (computer printouts of dry weather flow gaging results are included in Appendix A). The Neponset study area consultant has estimated a corresponding figure of 3.9 mgd, which was based on a summation of measured dry weather flow inputs to the Dorchester interceptor from major trunk sewers within their area. The interceptor was not monitored directly was because of excessive sediment deposits. For the purpose of preparing Figure V-5, the CDM-measured dry weather flow rate of 4.74 mgd was used, because it probably best represents the average daily dry weather flow rate from the Neponset study area when the following factors are considered:

- since there is an inherent inaccuracy in field gaging equipment, gaging at only one location eliminates the compounded error that results from adding results from several gages,

- the sediment bed depth in the interceptor upstream of the gaging location was frequently monitored to determine the bottom slope and to accurately measure the true depth of flow,

- velocity measurements were performed during flow monitoring in order to measure the roughness coefficient (Manning's "n" value) for use in flow calculations.

As shown on Figure V-5, the above analysis results in a total current estimated average dry weather flow of 65.61 mgd at the Columbus Park headworks. Again, this is based on a measured flow from the Neponset study area, calculated flows from the Dorchester Bay study area (verified by field measured results as discussed below) and estimated total flows from the Inner Harbor study area, proportioned by CDM. From analysis of Columbus Park headworks' flow records between October 1978 and January 1979, which includes the period when all field gaging was performed, the average dry weather flow ranged between 51 and 69 mgd with an average of about 60 mgd. Thus, the resulting average dry weather flow rate of 65.61 mgd shown on Fig. V-5 is within 9 percent of the average measured dry weather flow at the headworks. Considering that several sources of input data were used in the preparation of the flow balance shown on Figure V-5, and that the resultant flow is within the range of measured flows at the headworks, the flow balance is considered to be representative of actual field conditions, within a degree of accuracy acceptable for this project.

## Existing Dry Weather Flow Quality

The quality of dry weather flow within the Dorchester Bay study area was determined at the monitoring and sampling locations listed on Table V-3 (Figures V-1, V-2, and V-3) with the exception of DWF-8 (Dorchester Interceptor at McKone St.), where flow quantification only was performed. In accordance with Table B of the Quality Assurance Plan the water quality parameters analyzed were as follows:

- o Dissolved Oxygen (DO)
- o pH
- o 5 Day Biochemical Oxygen Demand ($BOD_5$)
- o Total Suspended Solids (TSS)
- o Total Coliform Bacteria (TC)
- o Fecal Coliform Bacteria (FC)
- o Heavy Metals - Cadmium (Cd)
  Chromium (Cr)
  Lead (Pb)
  Mercury (Hg)
  Zinc (Zn)

A summary of the analytical results obtained from the dry weather flow sampling program is shown on Table V-4, along with average Deer Island Wastewater Treatment Plant influent water quality parameters between January 1977 and August 1978. As shown, the average total coliform count at the Dorchester Bay study area sampling locations is of the same order of magnitude as the Deer Island influent count; however, the study area's average fecal coliform count is about one order of magnitude greater than Deer Island's. Average study area BOD and TSS concentrations were somewhat lower than corresponding Deer Island influent values. However, the average BOD concentrations were higher in samples taken from the two study area interceptors than in those taken from upstream combined sewers. Except for one sampling location (DWF-1; K St. at Marine Rd.), all average TSS concentrations, including those of the two interceptors, were lower than the average Deer Island concentration.

Differences in sampling and laboratory analysis may contribute to the discrepancy in study area and Deer Island influent contaminant concentrations. Also, the wide range of values reported are reflective of the daily changes in pollutant concentrations. As expected, results showed that the concentrations of contaminants were lower during early morning hours at all seven sampling locations, due to the higher proportion of infiltration within the waste stream. Averaging the lower, early morning contaminant concentrations with the higher concentrations that occur during the normal wastewater discharge periods, resulted in lower overall study area averages as compared to average Deer Island influent

TABLE V-4. RESULTS OF DRY WEATHER FLOW SAMPLING PROGRAM FOR THE DORCHESTER BAY STUDY AREA *

| Location Designation(1) | Location Description(1) | pH (ave.) | D.O. mg/l (ave.) | $BOD_5$, mg/l | | TSS, mg/l | | Tot. Coliform-No.x$10^6$/100ml | | Fecal Coliform-No.x$10^6$/100ml | | Heavy Metals (Total), mg/l | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | Range | Ave. | Range | Ave. | Range | Ave. | Range | Ave. | Cd | Cr | Pb | Hg | Zn |
| DWF-1 | K. St. at Marine Rd. | 7.4 | NR | 17-110 | 80 | 46-305 | 170 | NR | NR | 1.3-8.6 | 4.3 | 0.003 | 0.004 | 0.030 | ≤ 0.0002 | 0.070 |
| DWF-2 | So. Boston Interceptor near Mt. Vernon St. | 6.7 | 2.0 | 100-460 | 230 | 16-310 | 110 | 5.0-24.0 | 12.0 | 1.1-3.8 | 2.0 | 0.020 | 0.040 | 0.093 | 0.0006 | 0.080 |
| DWF-3 | Dorchester Interceptor near Morrissey Blvd. | 6.9 | 1.7 | 100-250 | 180 | 21-120 | 72 | 6.5-39.0 | 22.0 | 2.5-4.9 | 3.8 | 0.003 | ≤0.004 | 0.050 | ≤0.0002 | 0.065 |
| DWF-4 | Bay St. at Maryland St. | 6.8 | 2.9 | 6-110 | 38 | 8-100 | 57 | 0.3-25.0 | 41.0 | 0.07-4.7 | 2.0 | NR | NR | NR | NR | NR |
| DWF-5 | Geneva Ave. at Tonawanda St. | 6.4 | 5.0 | NR | NR | 16-55 | 42 | 2.0-14.0 | 7.1 | 0.25-1.5 | 0.79 | 0.002 | ≤0.004 | 0.015 | 0.0004 | 0.035 |
| DWF-6 | Gibson St. at Dorchester Ave. | 6.6 | 2.0 | 40-94 | 68 | 18-115 | 74 | 4.8-50.0 | 20.0 | 0.5-2.8 | 1.6 | 0.0007 | ≤0.004 | 0.018 | 0.015 | 0.010 |
| DWF-7 | Victory Rd. at Neponset Ave. | 6.9 | 2.1 | 42-100 | 72 | 8-110 | 68 | 16.0-80.0 | 27.0 | 0.37-5.2 | 2.8 | 0.002 | 0.004 | 0.022 | ≤0.0002 | 0.045 |
| TOTAL AVERAGE | | 6.8 | 2.6 | | 111 | | 85 | | 21.5 | | 2.5 | 0.005 | 0.060(2) | 0.038 | 0.0028(2) | 0.051 |
| Deer Island WWTP Influent (Ave. Jan. '77 to Aug. '78) | | NR | NR | | 148 | | 166 | | 36 | | 0.3 | NR | NR | NR | NR | NR |

NOTES:

(1) Sampling locations are shown on Figures V-1, V-2 and V-3.

(2) Average includes the values as shown which are reported as less than or equal to ( ≤ )

NR = Not Reported

*For sampling dates, refer to Table V-1

levels. Since the Deer Island Wastewater Treatment Plant serves an extremely large tributary area (75,400 acres), consisting of a multitude of industries, institutions, hospitals and other sources of continuous wastewater discharges, the influent contaminant levels tend to be more consistent throughout the plant's diurnal flow pattern, because infiltration is a lower proportion of the plant influent waste stream. This results in relatively higher average contaminant levels compared to those occurring within the Dorchester Bay study area (2,894 acres), which is a small portion of the plant's tributary area having no major sources of continuous daily wastewater discharges.

### Dry Weather Discharges through Combined Sewer Overflow Outlets

As discussed in Chapter IV, dry weather discharges from CSO outlets are usually caused by blockages in sewers and/or regulators due to excessive sediment buildup or other obstructions. The blockage prevents flow in a regulator from entering the dry weather flow connection to an interceptor or trunk sewer, and causes the flow to build up and spill over into an overflow conduit, from which it is discharged directly to Dorchester Bay. This type of problem is generally not a structural deficiency, but rather one which can be corrected by periodic maintenance.

In some instances, however, dry weather overflows may occur due to insufficient height of an overflow weir in a regulator. In this case, peak dry weather flows may result in a depth of flow greater than the weir height, resulting in the discharge of a portion of the flow to the overflow conduit and Dorchester Bay. Additional sources of dry weather discharges discovered during our field investigations are direct connections which convey sanitary flows, oil wastes and other types of waste to overflow conduits leading directly to Dorchester Bay. Structural changes, such as increasing the weir height or rerouting improper connections to the wastewater collection system, are necessary to remedy these type of problems.

During the field inspection program, several occurrences of direct dry weather discharge were discovered in the Dorchester Bay area, five of which were due to blockages within regulators and/or their dry weather flow connection to a trunk sewer. Attempts were made by the Boston Water and Sewer Commission and CDM field personnel to clean all known blocked regulators, but in some cases, the blockages have reoccurred. The locations where discharges were observed, and the problems associated with each, are summarized in Table V-5 (a detailed discussion of these problem areas is presented in Chapter IV). It should be noted that this table represents only those cases discovered in the field and that the total number of instances of direct dry weather discharge occurring within the Dorchester Bay study area is unknown.

TABLE V-5

SUMMARY OF KNOWN DIRECT DRY WEATHER DISCHARGES TO DORCHESTER BAY

| LOCATION | OBSERVED PROBLEM | ESTIMATED TRIBUTARY AREA CONTRIBUTING DWF (acres) | ESTIMATED DWF DISCHARGED TO DORCHESTER BAY* (gpd) | CSO DISCHARGE OUTLET NUMBER |
|---|---|---|---|---|
| South Boston - E. First Street, between Farragut Road and P Street | Blocked regulator | 10 | 24,500 | BOS - 080 |
| South Boston - E. Second Street, near Farragut Road | Low Overflow weir | 37 | unknown | BOS - 080 |
| South Boston - Farragut Road, near E. Sixth Street | Blocked regulator | 74 | 396,300 (plus oil) | BOS - 081 |
| Dorchester - Savin Hill Avenue, at Auckland Street | Blocked regulator | 9 | 58,200 | BOS - 089 |
| Dorchester - Bay Street, at Auckland Street | Blocked regulator | 65 | 283,500 | BOS - 089 |
| Dorchester - Auckland Street, near Hoyt Street (Dorchester Interceptor) | Overflow weir on interceptor** | 1,589 | unknown | BOS - 089 |
| Dorchester - Leroy Street, near Ditson Street | Blocked regulator | 1 | 11,400 | BOS - 090 |
| Dorchester - Storm drain near Victory Road and Morrissey Blvd. | Oil observed in overflow conduit (possible direct connection) | - | 0 (oil only) | BOS - 090 |
| | | TOTAL | 773,900 | |

*Represents estimated dry weather flow, including infiltration.
**Potential problem only - flow observed to be within two inches of weir crest during dry weather.

If it is assumed that, in the case of an obstructed dry weather flow connection, all dry weather flow entering the regulator is diverted continuously into the overflow conduit, estimates can be made of the quantity of dry weather discharge that occurred at each observed site prior to remedial action. These estimates are derived from the population of the area tributary to the regulator, and per capita wastewater production (calculated as discussed in the preceding section). These estimated quantities and the CSO outlet to which they discharge are also included in Table V-5. Intermittent dry weather flows discharging over low overflow weirs, or quantities of other types of wastes discharged continuously, such as oil, could not be estimated.

In summary, it is evident that dry weather flow discharges into Dorchester Bay waters as a result of structural or maintenance-related problems associated with the existing sewer system. Because of their highly polluted nature, dry weather flow discharges have a severe impact on the quality of the receiving water. In fact, these discharges are most likely a major source of continuous high coliform levels present in the Bay. (See Chapter IX of this report for a detailed discussion of water quality impacts of dry weather discharges.) Thus, the elimination of dry weather discharges from combined sewer overflow outlets should be of primary concern, and would most likely improve the water quality of Dorchester Bay.

# CHAPTER VI - WET WEATHER FLOWS

## Field Inspection, Gaging and Sampling Program

Gaging and sampling of wet weather flows and combined sewer overflows, comprised part of the overall field program. Continuous depth of flow monitoring of wet weather flows within the collection system, provided valuable information on quantities of combined sewer overflows, and on general system response during storm events. Quality data obtained from wet weather flow samples was used to assess the impacts of combined sewer overflows on Dorchester Bay. Both quality and quantity information was collected for use in computer modeling, and this data further serves the purpose of providing a means of verifying the results obtained by the model simulation.

The formal field program was conducted from July 17 to November 22, 1978. However, because of the limited hydraulic data generated during the formal field program (due to a lack of significant rainfall), additional field monitoring was conducted in January and September of 1979. Field inspection of the major combined sewer overflow conduits and outlets was performed in the initial stages of the program to determine suitable monitoring station locations, and to identify potential problem areas. Gaging and sampling equipment was subsequently installed at the selected locations, and manually activated during significant storm events.

Description of Equipment. Three Manning portable automatic discrete samplers (Model No. S-4040-2) were used throughout the field program to collect samples from combined sewer overflows and separate storm drains at the selected locations and at specified time intervals during storm events. The sampling unit utilizes a pressure-vacuum method which draws samples rapidly enough to obtain a representative level of suspended solids in the samples, conforming to EPA guidelines (EPA Report - EPA-R2-73-261, "An Assessment of Automatic Sewer Flow Samplers"). The guidelines specify that sample transport velocities be maintained above a minimum of 3 ft/sec; the Manning samplers can meet this criteria at 15 ft of suction head. Each unit is equipped with a 3/8-in. diameter, 25-ft long suction hose, 24-500-ml polypropylene sampling bottles, a rechargeable 12-volt battery and a suspension harness. A positive purge cycle is initiated before and after each sample is taken, which clears the inlet hose and prevents cross-contamination of samples. A time proportional cycle control on each unit allows the selection of time interval between samples. The sample volume is adjustable from 25 to 500 ml, within 0.5 percent accuracy.

Wet weather flow measurement was accomplished by use of Manning dippers and Bristol gages. Velocity measurements were also taken during wet

weather events at various locations using a Gurley current meter. Refer to Chapter V for a description of all flow measurement equipment.

Gaging and Sampling Locations. Locations for monitoring and sampling were established on combined sewer overflow conduits downstream of major drainage basins, where significant amounts of overflow could be expected during a wet weather event. Gaging and sampling equipment was also installed on separate storm sewers, to obtain information on the quantity and quality of storm water runoff (without contribution from sanitary flow). These sites were subject to the same criteria that were applied to the selection of dry weather flow monitoring locations, regarding conduit characteristics, hydraulic controls, accessibility, etc. (see Chapter V).

A summary of the wet weather monitoring and sampling program is presented in Table VI-1. Included in the table are locations where data was collected, storm events when data collection was performed, and a description of the gaging and sampling work performed at each location. All locations are shown on Figures V-2, and V-3 which are contained in Chapter V. As shown on Table VI-1, the storm events of 6 and 14 October 1978 and 8 and 25 January 1979 were the four major events that provided substantial wet weather flow quantity and quality information. Data was collected at a total of nine locations within the Dorchester Bay study area during the field program.

Gaging and Sampling Procedures. Wet weather flow gaging and sampling was conducted in accordance with the 16 August 1978 "Quality Assurance Plan/ Monitoring and Sampling Guidelines" prepared for the Dorchester Bay study area. The general implementation of the sampling program for a typical storm event proceeded as follows:

- Four hours prior to the start of an anticipated storm, a decision was made to mobilize based on weather service information; field and laboratory personnel were then notified.

- The field crew proceeded to each of the three automatic sampler locations, inspected the unit and added ice to the central compartment to ensure proper cooling. The unit was set for the required sampling frequency, and turned on at the first evidence of significant flow in the conduit.

- During the first 60 minutes of the overflow event, 1500 ml samples (3-500 ml sampler bottles each) were taken at intervals of 7.5 minutes. Dissolved oxygen, temperature, and pH readings were taken of the flow in the conduit, using a portable meter equipped with a probe.

- At the end of the first hour, samples were removed from the unit and placed in an iced cooler. New sample bottles were

TABLE VI-1. SUMMARY OF THE WET WEATHER FLOW MONITORING AND SAMPLING PROGRAM FOR THE DORCHESTER BAY STUDY AREA

| Location Designation (1) | Location Description (2) | Storm Events When Data Collection Was Performed | | | | | | | | Description of Flow Gaging Performed | Description of Sampling Performed | Comments |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | 10-4-78 | 10-6-78 | 10-14-78 | 11-17-78 | 1-8-79 | 1-25-79 | 5-25-79 | 9-6-79 | | | |
| SS-1 | Josephine St. near Ditson St. | X | X | X | | | | | | Continuous - Bristol gage with weir | Discrete samples taken with automatic sampler | |
| SS-2 | Parkman St. | | | | X | | | | | Continuous - Manning dipper installed, however no significant flow occurred | Grab samples taken in shallow (2" deep ±) flow | Storm of 11-17-78 (0.49" of rain over 17 hrs) did not generate CSO's or significant storm drain flowrates at gaging locations. |
| SSS-1 | Neponset Ave. at Garvey Playground (Pine Neck Creek Storm Drain) | | | | X | | | | X | None | Grab sample taken in shallow (2-4" deep±) flow | |
| CSO-1 | Malibu Beach on Overflow Conduit Leading to Outlet BOS-089 | | | | | X | X | X | X | Depth of flow with surveyor's rod. Velocity measurements with Gurley meter | Grab samples taken | Since formal field program ended on 11-22-78, continuous monitoring and sampling could not be performed. |
| CSO-2 | Morrissey Blvd. on Overflow Conduit Leading to Outlet BOS-090 | | | | | X | X | X | X | Depth of flow with surveyor's rod. Velocity measurements with Gurley meter | Grab samples taken | |
| CSO-3 | Victory Rd. near Houghton St. | | | X | | | | | | Continuous - Manning dipper | Discrete samples taken with automatic sampler | No equipment was in place during 10-4-78 and 10-6-78 storms. Equipment from CSO-5 moved here 10-12-78. |
| CSO-4 | Geneva Ave. near Tonawanda St. | | X | X | | | | | | Continuous - Bristol gage | Discrete samples taken with automatic sampler | |
| CSO-5 | Dorchester Ave. near Gibson St. | | | | | | | | | Manning dipper installed however no overflow events occurred | Automatic sampler installed however no overflow events occurred | Equipment was in place during 10-4-78 and 10-6-78 storms, however no overflow occurred. Moved to CSO-3 on 10-12-78. |
| CSO-6 | CSO Outlet BOS-090 at Commercial Point | | | | | | | X | | Depth of flow with surveyor's rod | None performed | See comments for locations CSO-1 and CSO-2 |

Notes:

(1) SS = Separate storm sewer monitoring and sampling location.
SSS = Separate storm sewer sampling location.
CSO = Combined sewer overflow monitoring and sampling location.

(2) All locations are shown on Figures V-2 and V-3 in Chapter V.

added to the sampler case and the required sampling interval set for the next phase of the program. Samples were taken every 15 minutes for the next 2 hours, and every hour thereafter up to 6 hours or to the end of the overflow event, whichever was shorter.

- At some locations, grab samples were taken manually by field personnel at regular intervals. Dissolved oxygen, temperature and pH measurements of the grab samples were made on-site immediately after the sampling was performed.

- Discrete and grab samples (in ice coolers) from all sampling locations were collected at the end of each sampling sequence and transported to the CDM laboratory for analysis.

Wet weather flow samples were analyzed for BOD , TSS, VSS, settleable solids, total and fecal coliforms, chlorine demand, chlorides, five heavy metals (cadmium, chromium, lead, mercury and zinc) total phosphorus, total Kjeldahl nitrogen, oil and grease, volatile organics, pesticides and PCB's. Results of these analyses will be discussed in a later section.

Gaging and Sampling Events. As a result of unusually dry weather conditions, only two significant storms, which occurred on 6 and 14 October, 1978, produced combined sewer overflows during the formal field program (ending 22 November 1978). The actual overflow durations at the sampling locations were relatively short, due to the lack of prolonged high intensity rainfall during these storms. A relatively smaller storm (0.24 in total rainfall) on 4 October produced sufficient runoff to be monitored and sampled at the Josephine Street storm drain station, but failed to cause combined sewer overflows at the installed CSO monitoring locations.

The 5-hour storm of 6 October resulted in a total rainfall of 0.85 in. As shown in Table VI-1, continuous flow gages and automatic samplers were installed for operation on two combined sewer overflow conduits (Geneva Avenue, CSO-4 and Dorchester Avenue, CSO-5) and on the separate storm drain on Josephine Street (SS-1). However, the overflow event at Geneva Avenue lasted only one hour, and no overflow was recorded at the Dorchester Avenue location. Quality and quantity data were obtained at the Geneva Avenue and Josephine Street gaging locations for this storm. The gage at Dorchester Avenue was relocated to the Victory Road location (CSO-3) on 12 October 1978.

The storm of 14 October produced 1.01 inches of rain over a period of 10 hours. Quantity and quality data were obtained at the Geneva Avenue and Victory Road stations, from overflow events that lasted 20 minutes and 4.5 hours, respectively. Additional information on stormwater was also collected at the Josephine Street station. It should be noted that, despite the lack of abundant field monitored CSO information during the formal field program (both quality and quantity), some information was obtained on the hydraulic response of the system at various locations

during some minor wet weather events. This information was obtained at dry weather flow gaging stations that were in place and operating during wet weather events. Data included information on the wet weather response of the Dorchester and South Boston interceptors. It should be noted that the gaging periods shown on the appended dry weather flow hydrographs were selected to best represent flows during dry weather, and may not necessarily reflect the entire period gaged. Also, during the 14 October storm, velocity and depth-of-flow measurements were taken at specific locations within the system, including two major overflow conduits (those leading to Outlets BOS-089 and BOS-090).

The formal field monitoring and sampling program ended with the removal of all field equipment on 22 November 1978. However, in order to supplement the limited amount of data obtained during the formal program, two additional wet weather events were monitored: a 35-hour, 1.95-in. rainfall event on 7 and 8 January 1979 and a 30-hour, 2.63-in. rainfall event which occurred from 24 to 26 January 1979. A field crew was dispatched to obtain periodic depth of flow and conduit velocity measurements, and grab samples (for quality monitoring) on the two major CSO conduits in the Dorchester portion of the study area. These conduits, which serve 62.8 percent (1,814 acres) of the study area, are:

1) The 96-in. x 120-in. conduit along Malibu Beach, which serves to convey CSO's to Outlet BOS-089 at Fox Point

2) The 144-in. x 144-in. conduit extending from near Victory Road to Outlet BOS-090 at Commercial Point.

Five depth-of-flow and velocity measurements, and three grab samples, were taken at each conduit during the 7 and 8 January storm. During the 24 to 26 January storm, depth of flow, velocity measurements and grab samples were taken four times during the storm on conduit (1), and six times during the storm on conduit (2). This additional information was valuable for calibration of the models, and served as a supplement to the formal field gaging and sampling program.

Since additional field monitoring and sampling was considered necesary for both model calibration and CSO quality assessment, the field program was extended to include two additional CSO events in 1979. Depth-of-flow measurements were taken at three CSO locations in Dorchester (see Table VI-1) and one location on the Dorchester Interceptor (site DWF-3 on Figure V-1) during a short duration storm which occurred on May 25, 1979. The storm produced a total of 0.74 inches of intermittent precipitation, resulting in limited CSO quantification data. The data collected, however, was useful for calibration of the computer models.

Monitoring and sampling were also conducted on September 6, 1979 during a significant storm event (total precipitation of 1.08") at three locations: on each of the two major CSO conduits in Dorchester, described in (1) and (2) above, and on the Pine Neck Creek storm drain. Grab samples of combined sewer overflow and stormwater were collected and analyzed for total suspended solids, total and fecal coliform, volatile organics, organochlorine pesticides and polychlorinated biphenyls (PCB's). Results of these analyses are presented in the next section.

Wet Weather Flow Quality

The quality of wet weather flows was determined at the monitoring and sampling locations listed in Table VI-1 with the exception of CSO-5 (Dorchester Ave. near Gibson St.) where an overflow did not occur during monitored storm events, and CSO-6 (CSO Outlet BOS-090 near Commercial Point) where flow quantification only was performed. In addition, the quality of samples collected at dry weather flow monitoring locations DWF-1, DWF-2 and DWF-3 during wet weather events was also determined. The following water quality parameters were analyzed:

- pH
- Dissolved Oxygen (DO)
- Biochemical Oxygen Demand ($BOD_5$)
- Total Suspended Solids (TSS)
- Volatile Suspended Solids (VSS)
- Settleable Solids
- Total Coliform Bacteria
- Fecal Coliform Bacteria
- Heavy Metals - Cadmium (Cd)
  Chromium (Cr)
  Lead (Pb)
  Mercury (Hg)
  Zinc (Zn)
- Oil and Grease
- Volatile Organics
- Pesticides
- Polychlorinated Biphenyls (PCB's)

A summary of the analytical results obtained from the wet weather flow sampling program is displayed in Table VI-2. Data relating to levels of volatile organics, pesticides and PCB's are shown separately on Table VI-3.

The water quality parameters utilized for land-based and harbor modeling were $BOD_5$, TSS, and total and fecal coliform concentrations. Of these, total coliform concentration was the major quality parameter employed during the analysis of water quality impacts of CSO alternatives (see Chapter X). Based on the results of the wet weather sampling program, a total coliform concentration of $2 \times 10^6$/100 ml was selected to represent the average concentration of CSO's for large intensity storms similar to the two that were sampled during January, 1979 (both storm were approximately representative of a 1-year frequency storm).

TABLE VI-2 RESULTS OF THE WET WEATHER FLOW SAMPLING PROGRAM FOR THE DORCHESTER BAY STUDY AREA

| Location Designation[1] | Location Description | pH (avg) | D.O. mg/1 (avg) | $BOD_5$ mg/1 (range) | TSS mg/1 (range) | VSS. mg/1 (range) | Settleable Solids mg/1 (range) | Total Colif. No.x$10^6$/100ml (range) | Fecal Colif. No.x$10^6$/100ml (range) | Total Heavy Metals mg/1 (avg.) Cd | Cr | Pb | Hg | Zn | Oil & Grease mg/1 | Alkalinity mg/1 (range) | TP mg/1 (range) | TKN mg/1 (range) | Cl Demand mg/1 (range) | $Cl^-$ mg/1 (range) |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| CSO-1 | Malibu Beach on Overflow Conduit leading to Outlet BOS-089 | | | | 21.-220. | | | 0.54-3.0 | 0.047-0.59 | | | | | | 12. | | | | | |
| CSO-2 | Morrissey Blvd. on Overflow Conduit leading to Outlet BOS-090 | | | | 82.-1640. | | | 0.27-1.8 | 0.038-0.10 | | | | | | 460. | | | | | |
| CSO-3 | Victory Road near Houghton Street | 6.9 | 6.7 | 80.-120. | 31.-315. | 27.-205. | 0.6-16.5 | | | .002 | .006 | 0.18 | .0002 | 0.15 | | 61. | 3.8 | 20. | | |
| CSO-4 | Geneva Ave. near Tonawanda Street | 6.5 | 6.7 | 32. | 48.-310. | 20.-165. | .05-6.0 | | | .001 | .010 | 0.22 | .0002 | 0.11 | | | | | | |
| SS-1 | Josephine Street near Ditson Street | 6.7 | 6.8 | 8.-29. | 2.-230. | 2.-60. | .05-1.0 | | | .0015 | .008 | .245 | .0005 | .065 | | | 0.4-2.5 | 6.7-7.2 | | |
| SS-2[2] | Parkman Street | 6.9 | | 140.-200. | 84.-180. | 44.-140. | 1.5-15. | 0.32-8.0 | .004-3.2 | .008 | .0006 | .052 | | 0.38 | | 52. | 1.2 | 29. | 38.-45. | 40.-47. |
| SSS-1 | Neponset Ave. at Garvey Playground (Pine Neck Creek Storm Drain) | 6.8 | | 150. | 56. | 30. | 10.-12. | 1.2 | .012 | .012 | .015 | 0.17 | | 0.40 | 4. | 28.-92. | 2.2-0.85 | 6.7-8.5 | 31.-42. | 57.-780 |
| DWF-1[3] | South Boston Interceptor near Devine Way | 7.2 | | 130. | 220.. | | 0.35 | 20. | 0.13 | | | | | | | | | | 81. | |
| DWF-2[3] | Dorchester Interceptor near Morrissey Blvd. | 6.8 | | 260. | 250. | | 8.0 | 3.0 | 0.047 | | | | | | | | | | 92. | |
| DWF-8[3] | Dorchester Interceptor near McKone Street | 6.8 | | 170. | 105. | | 3.0 | 13. | 0.14 | | | | | | | | | | 99. | 150. |

Notes:

[1] Sampling locations are shown on Figures V-1, V-2 and V-3.

[2] Data for this location is not representative of separate storm water quality since the storm event generated low runoff volumes and the samples were taken in 2-4 inches of relatively stagment flow.

[3] These dry weather flow sampling locations were also sampled during wet weather events.

TABLE VI-3 RESULTS OF LABORATORY ANALYSES OF WET WEATHER SAMPLES FOR VOLATILE ORGANICS AND PESTICIDES

| Compound[1, 2] | Sampling Location (and Type) | | |
|---|---|---|---|
| | Overflow Conduit Leading to BOS-090, at Bradlees' Parking Lot (CSO) | Overflow Conduit Leading to BOS-089 at Malibu Beach (CSO) | Pine Neck Creek Storm Drain at Garvey Playground (SW) |
| **Volatile Organics[3]** | | | |
| Toluene | <10. | 70. | N.D. |
| Chlorobenzene | <10. | N.D. | N.D. |
| Methylchloride | N.D. | <10. | N.D. |
| 1,1,1-Trichloroethane | N.D. | <10. | N.D. |
| 1,1,2-Trichlorotrifluoroethane | -- | D. | -- |
| **Organochlorine Pesticides** | | | |
| Aldrin | <0.01 | <0.01 | <0.01 |
| Dieldrin | <0.01 | 0.1 | <0.01 |
| Endrin aldehyde | <0.1 | <0.1 | <0.1 |
| Endrin | <0.01 | <0.01 | <0.01 |
| Heptachlor | <0.01 | <0.01 | <0.01 |
| Heptachlorepoxide | <0.01 | <0.01 | 0.05 |
| P,P'-DDT | <0.02 | <0.02 | <0.02 |
| P,P'-DDE | <0.01 | <0.01 | <0.01 |
| P,P'-DDD | <0.01 | <0.01 | <0.01 |
| $\alpha$-endosulfan | <0.3 | 1. | 2. |
| $\beta$-endosulfan | <0.3 | <0.3 | <0.3 |
| $\alpha$-BHC | 0.04 | <0.01 | 0.01 |
| $\beta$-BHC | 0.1 | <0.02 | <0.02 |
| $\gamma$-BHC | <0.02 | 0.6 | 0.3 |
| $\delta$-BHC | <0.03 | <0.03 | <0.03 |
| Endosulfansulfate | 2. | 0.7 | <0.05 |

---

Notes:

[1] All values reported as $\mu$g/l (i.e. parts per billion)

[2] Samples were also quantitatively scanned for: PCB's, chlordane, toxaphene. None of these compounds were detected.

[3] Special notes on volatile organics:

ND = Not detected (compound not present at <10 $\mu$g/l detection limit)

D = Detected; compound not quantified, not specified on EPA list of priority pollutants.

<10.= Compound present; however, concentration not quantified at levels of <10. $\mu$g/l

All other volatile organics specified by EPA as priority pollutants were not detected.

# CHAPTER VII - COMBINED SEWER MODELING

## Introduction

Due to the complex and variable hydrologic, hydraulic, and water quality relationships associated with combined sewer overflows, mathematical simulation models are considered important tools and were used for analysis and planning of CSO facilities for the Dorchester Bay area. This chapter describes the land-based simulation models employed in the study. The study area is shown, for convenience, on Figure VII-1.

Combined sewer overflows result from the inability of the study area's existing sewer system to collect overland runoff plus dry weather flow during frequent wet weather conditions and to properly convey the total to the Columbus Park headworks. These overflows, and their associated water quality impacts, can be thought of as resulting from the following sequence of events:

- o a rainfall event occurs and produces runoff that is collected by a network of pipes and is combined with sanitary (dry weather) flow in the sewers;
- o the combined flow, in turn, is routed towards the sewer system interceptors which have finite flow capacity and, therefore, overflow into the bay (at outfalls);
- o the pollutant loads contained in the overflow are dispersed in the bay and thereby contribute to water quality degradation.

Because of the complexity of the physical processes associated with each link in the above sequence, simulation models were utilized to investigate and analyze the study area's long and short-term response to rainfall for variations in sewer system design and/or operation.

This chapter describes the simulation modeling tools used in the CSO facilities planning process for the Dorchester Bay area. The following explains how and why particular models were selected and explains each model in detail.

## Selected Modeling Tools

No single computer model could satisfy all the needs of a CSO facilities planning study. Selecting an integrated "package" of programs which would meet the needs of this facilities plan involved consideration of three related issues: (1) the nature and availability of input data associated with the Dorchester Bay study area; (2) the "simulation functions" that would supply the desired planning-level information for eventual decision making; and (3) the type of model output that would be most useful for this planning project.

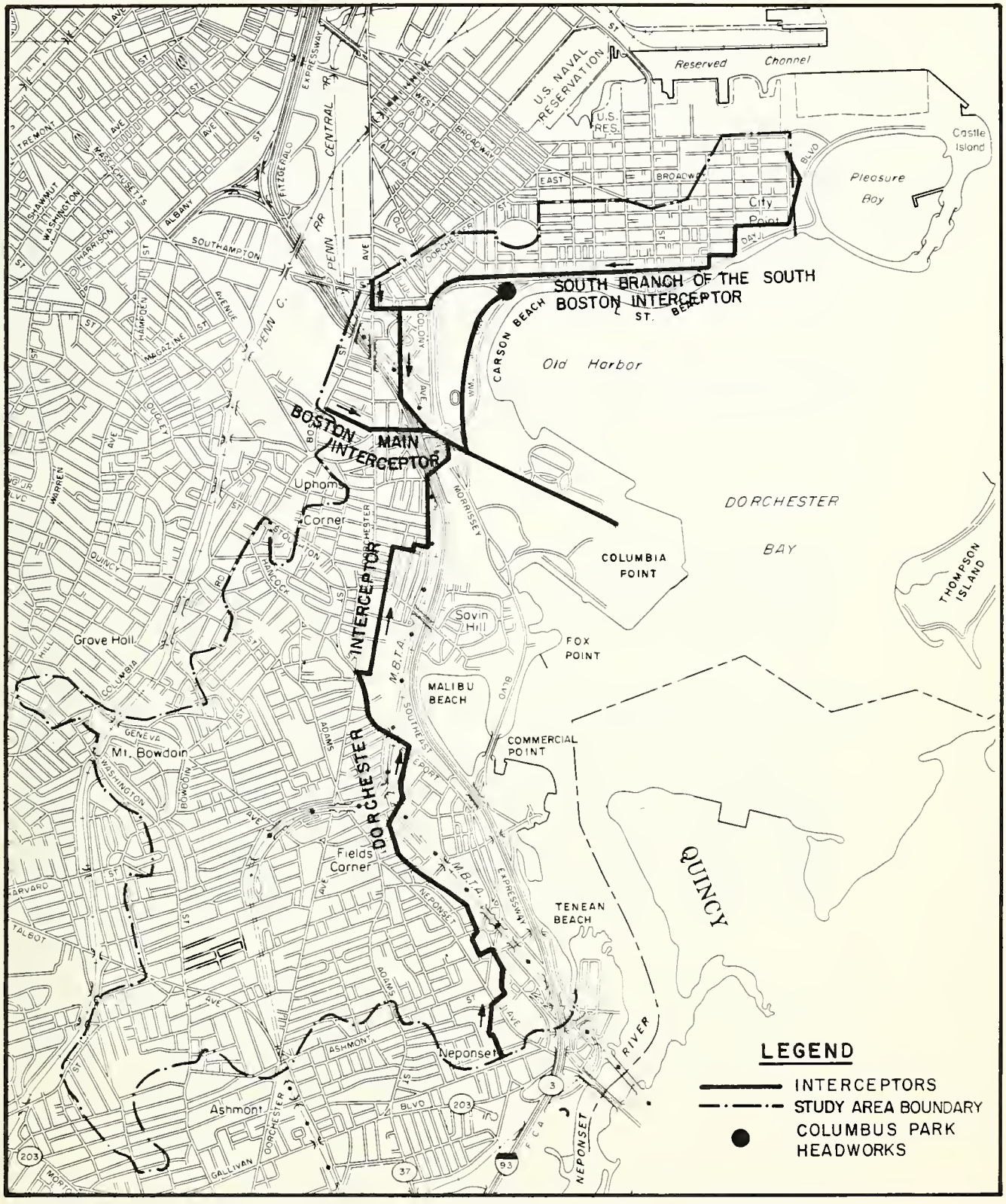


FIG. VII-1 DORCHESTER BAY STUDY AREA

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Figure VII-2 illustrates these related issues for the three simulation models (computer codes) which were selected for this work. Simulation capability is required to generate annual statistics of overflow quantity and quality to be used as input to the "Harbor Model" for the long-term assessment of CSO impacts on water quality in Boston Harbor (Chapter VIII). The same statistical capability is used for "screening" possible CSO alternatives for the mitigation of overflow problems (Chapter X). SEMSTORM meets these needs. Also, a capability to study the short term hydraulic sewer system response to "design storm" rainfall events is needed for the detailed evaluation of those alternatives which appear to be attractive, based on the screening process (Chapter XI). The RUNOFF and TRANSPORT "blocks" of the SWMM (Stormwater Management Model) model are used for this purpose.[1]

Figure VII-3 illustrates in schematic form how the three selected models actually interrelate to form a single "package" for the purpose of the Dorchester Bay area CSO facilities planning process. The remainder of this chapter documents each model's use in detail.

## The RUNOFF Model

The RUNOFF model was employed in this facilities plan to simulate the overland flow which results from single rainfall events on relatively small sub-catchments within the study area. The model simultaneously routes this flow through conveyance conduits to provide input for the TRANSPORT modeling work.

Conceptually, the RUNOFF model operates as shown schematically on Figure VII-4. The physical system is modeled as an inclined plane generating surface runoff. Also modeled is a pipe that collects the surface drainage and dry weather flow and conveys it downstream. For both the overland flow (surface runoff) and pipe conveyance (pipe routing) computation, the model solves the momentum and continuity relationships of flow using the kinematic wave approximation which assumes that the friction slope is equal to the land surface or pipe slope, respectively. For details associated with the kinematic wave theory and its use in the RUNOFF model, the reader is referred to the program documentation (see References).

Calibration. As can be seen from Figure VII-4, to generate the overflow hydrograph associated with a particular drainage sub-catchment, RUNOFF requires a rainfall hyetograph. The process of model calibration involves using RUNOFF with a hyetograph that corresponds to a period when field measured outflow hydrographs are available, and attempting to refine model parameters so that both simulated and observed hydrographs coincide reasonably well.

Figures VII-5 and VII-6 present hyetographs for the storms which occurred on August 6 and October 6, 1978, respectively. The two "calibration" areas shown on Figure VII-7 were field monitored, and the outflow hydrographs which correspond to the storms were recorded. These two

[1] Throughout the text, for clarity, the terms RUNOFF and TRANSPORT will be used as if they were separate, independent models.

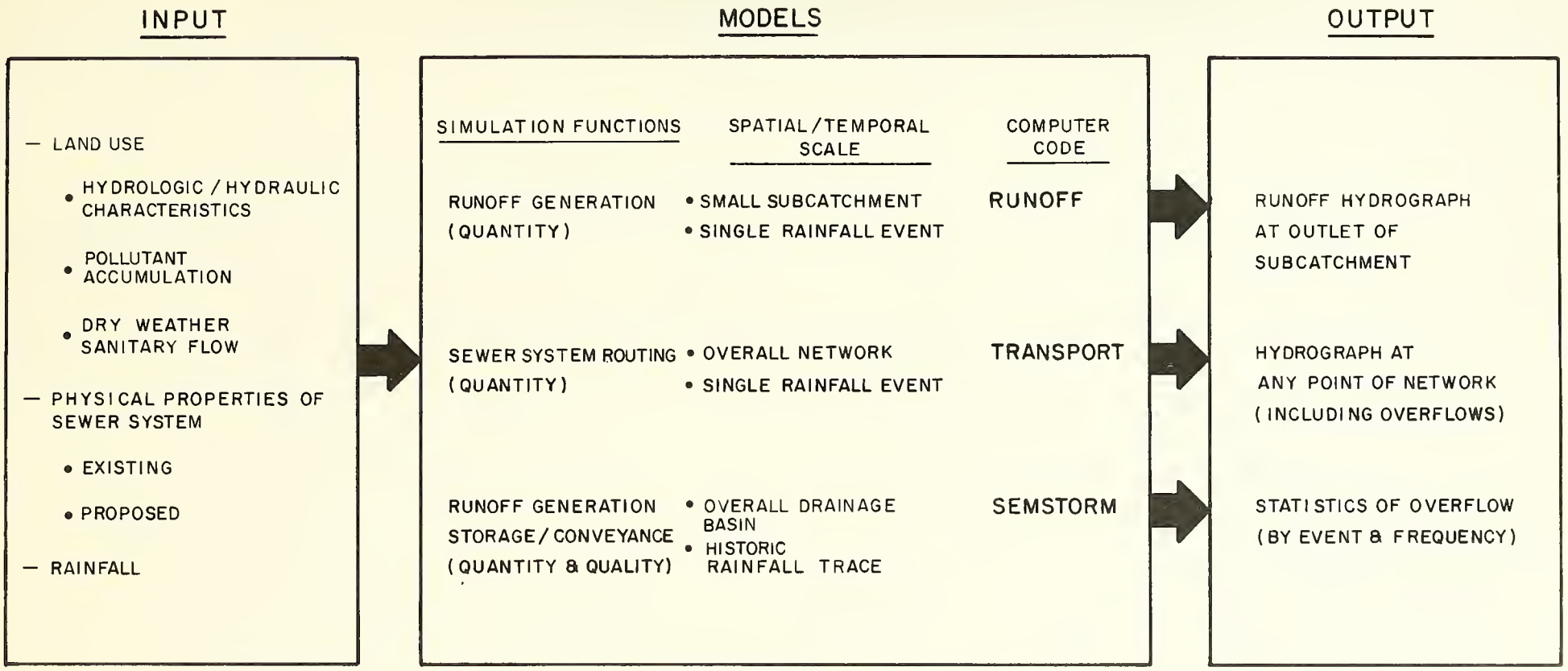


FIG. VII-2 COMPUTER MODELING OF STUDY AREA: INPUT, SIMULATION FUNCTIONS, COMPUTER CODE AND OUTPUT

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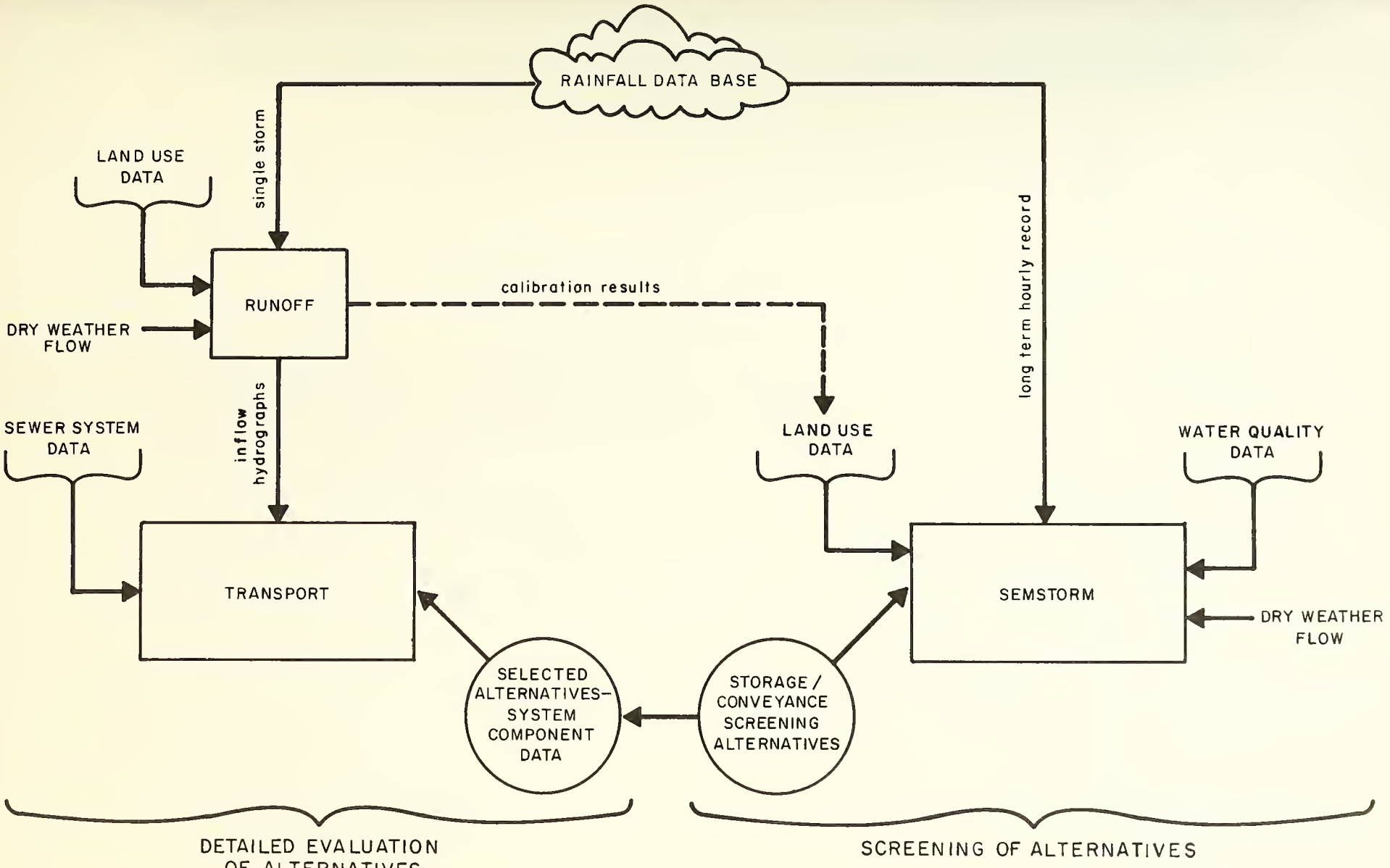


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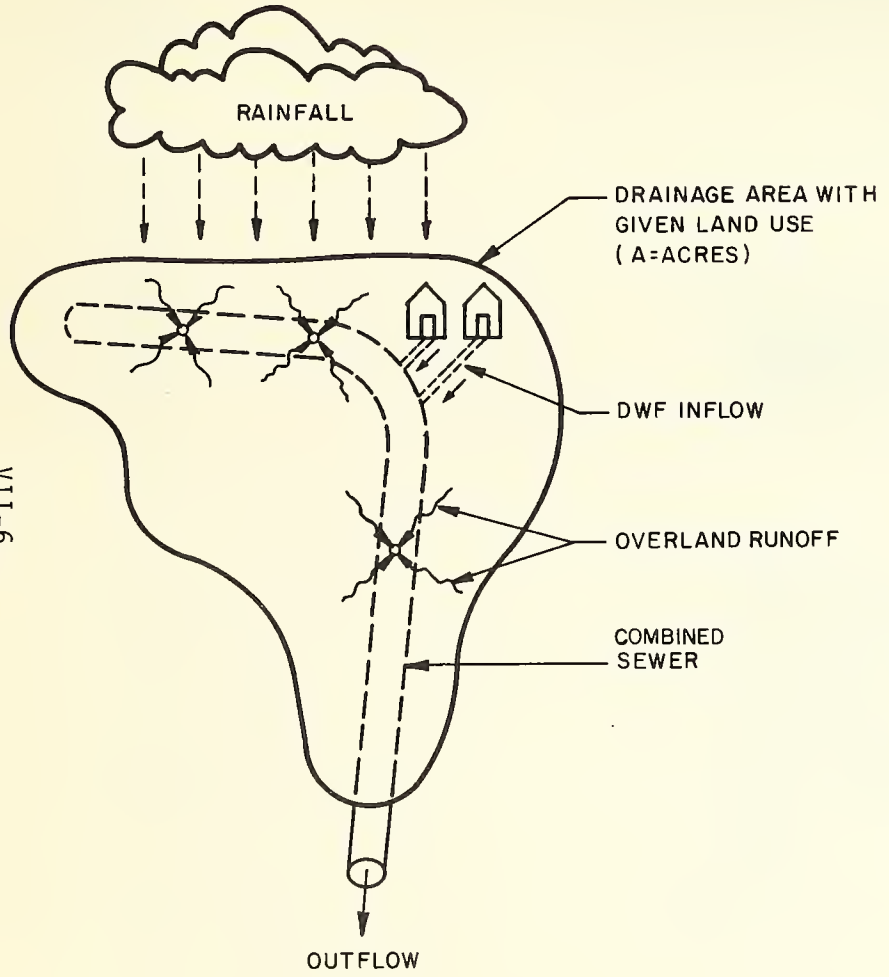
FIG. VII-3 INTERRELATION OF MODELING COMPONENTS

PHYSICAL SYSTEM




"RUNOFF" CONCEPTUALIZATION

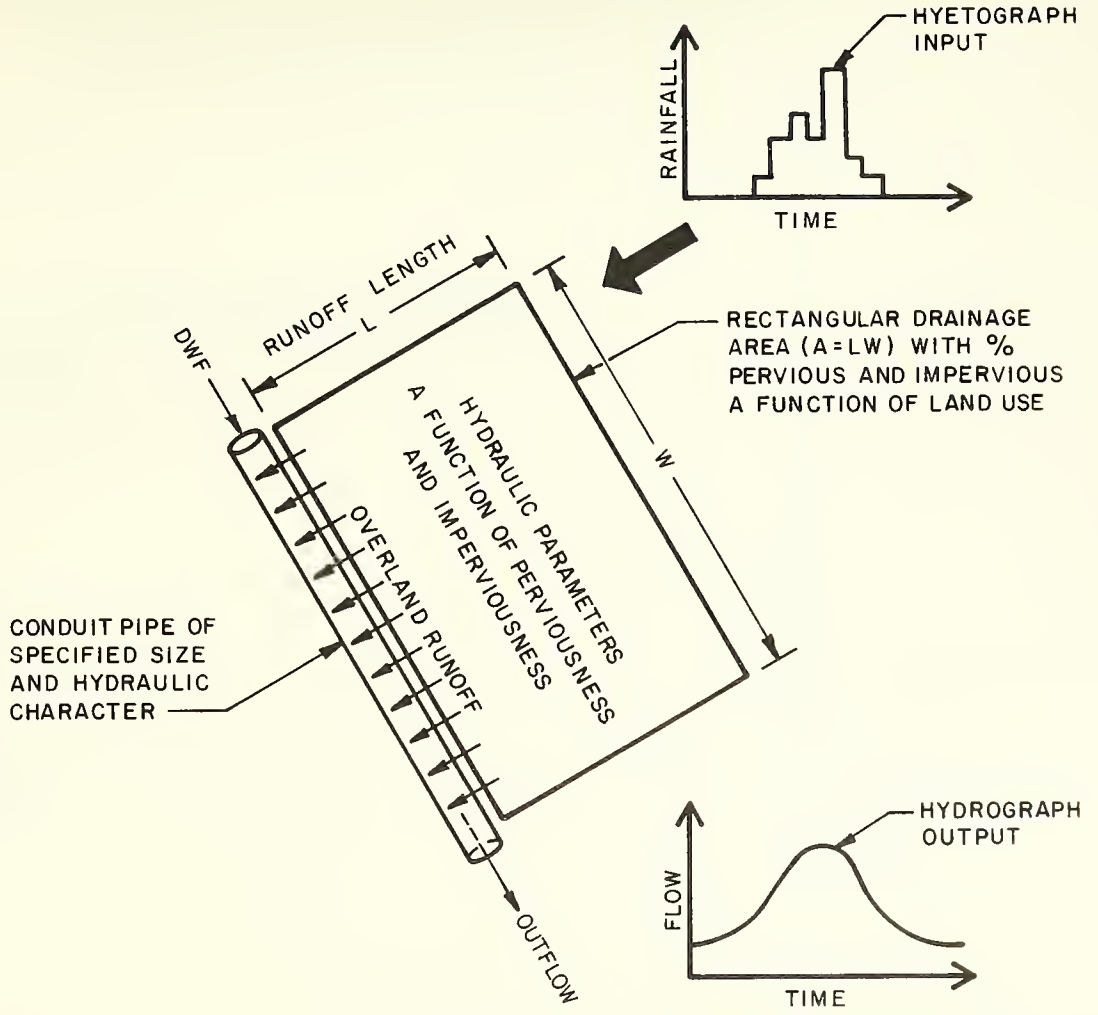



FIG. VII-4 THE RUNOFF MODEL'S MATHEMATICAL CONCEPTUALIZATION OF A PHYSICAL DRAINAGE AREA

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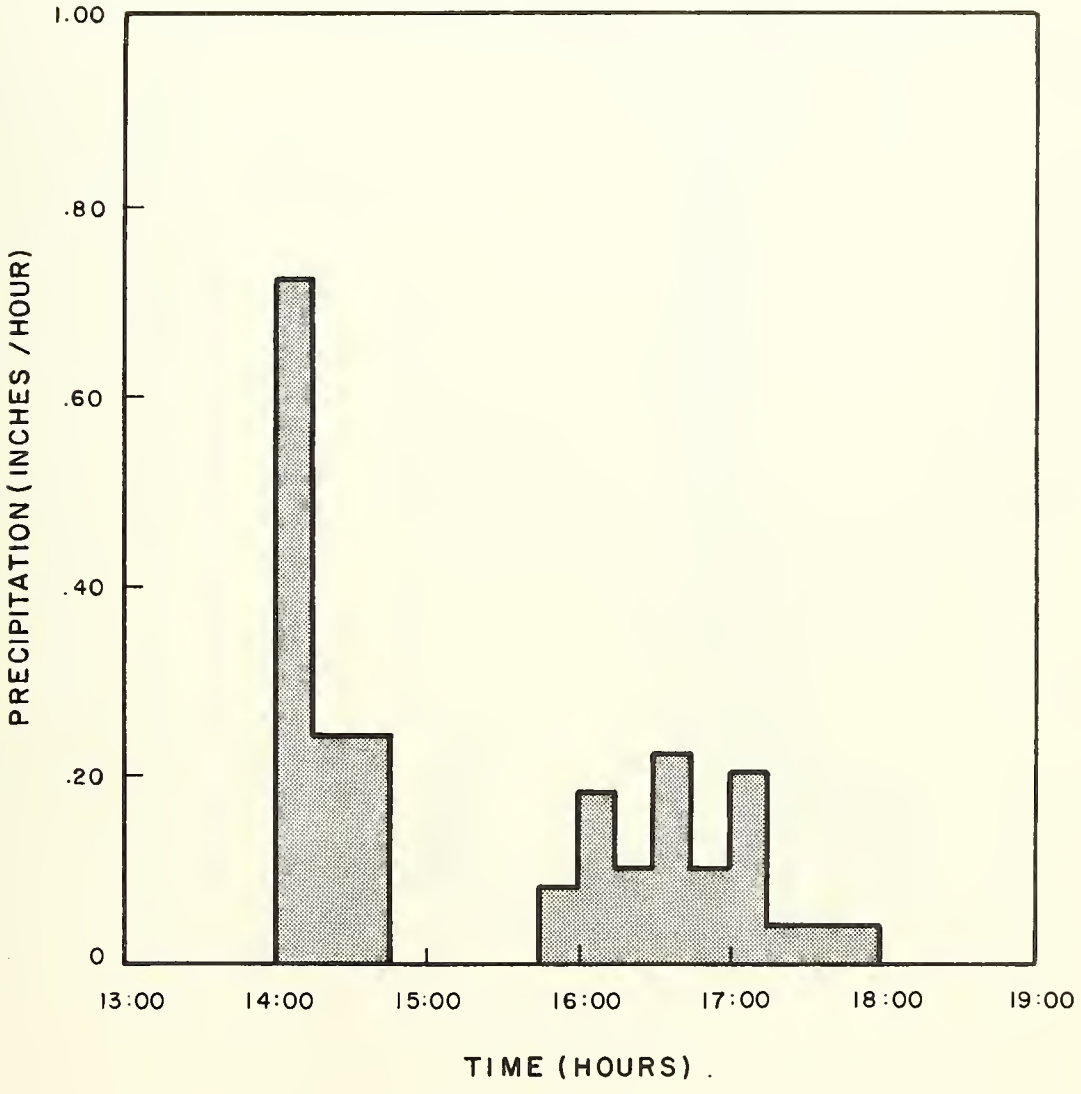


FIG. VII-5 HYETOGRAPH FOR STORM OF AUGUST 6, 1978
(BLUE HILLS DATA)

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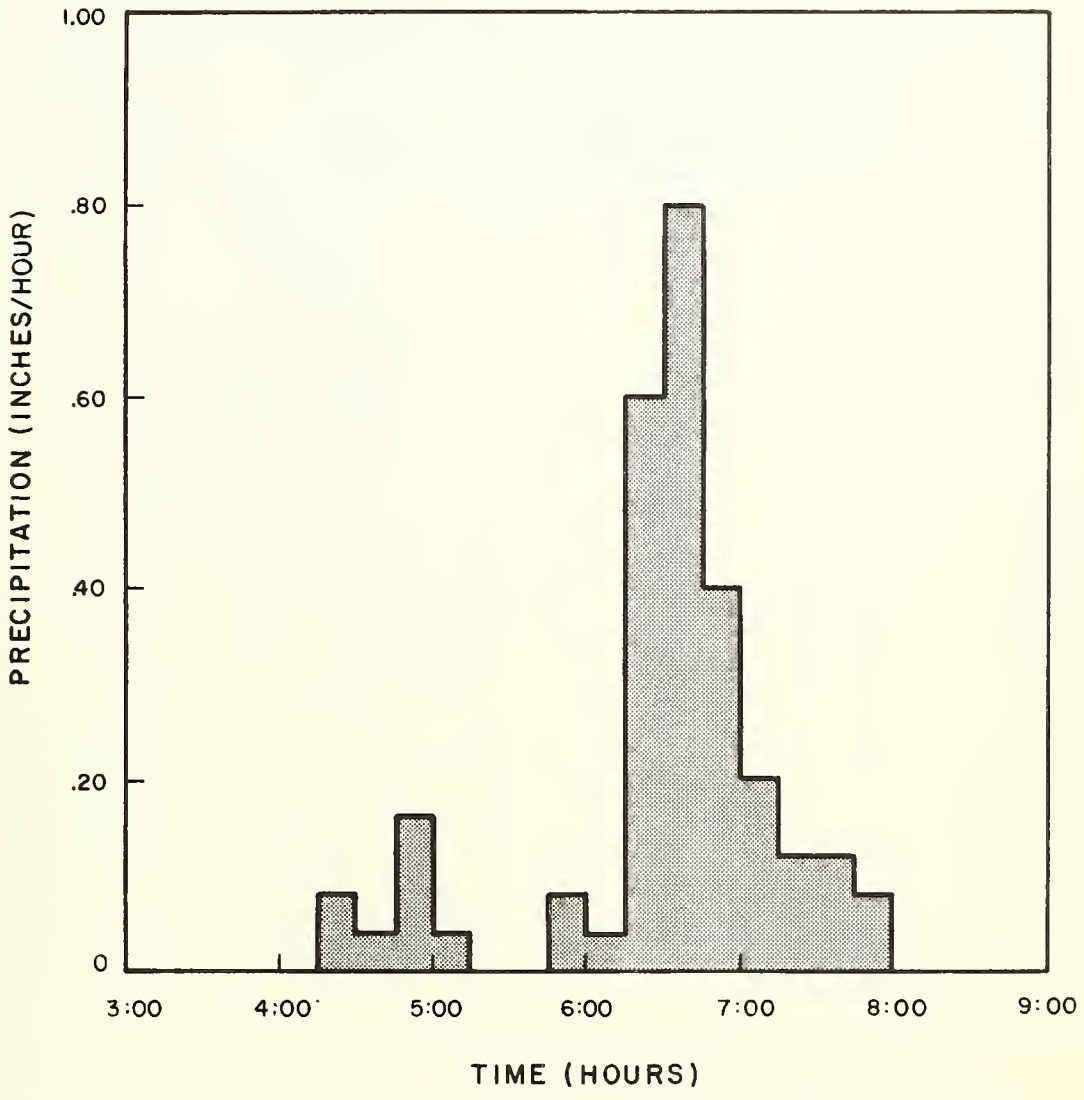


FIG. VII-6 HYETOGRAPH FOR STORM OF OCTOBER 6, 1978
(LOGAN AIRPORT DATA)

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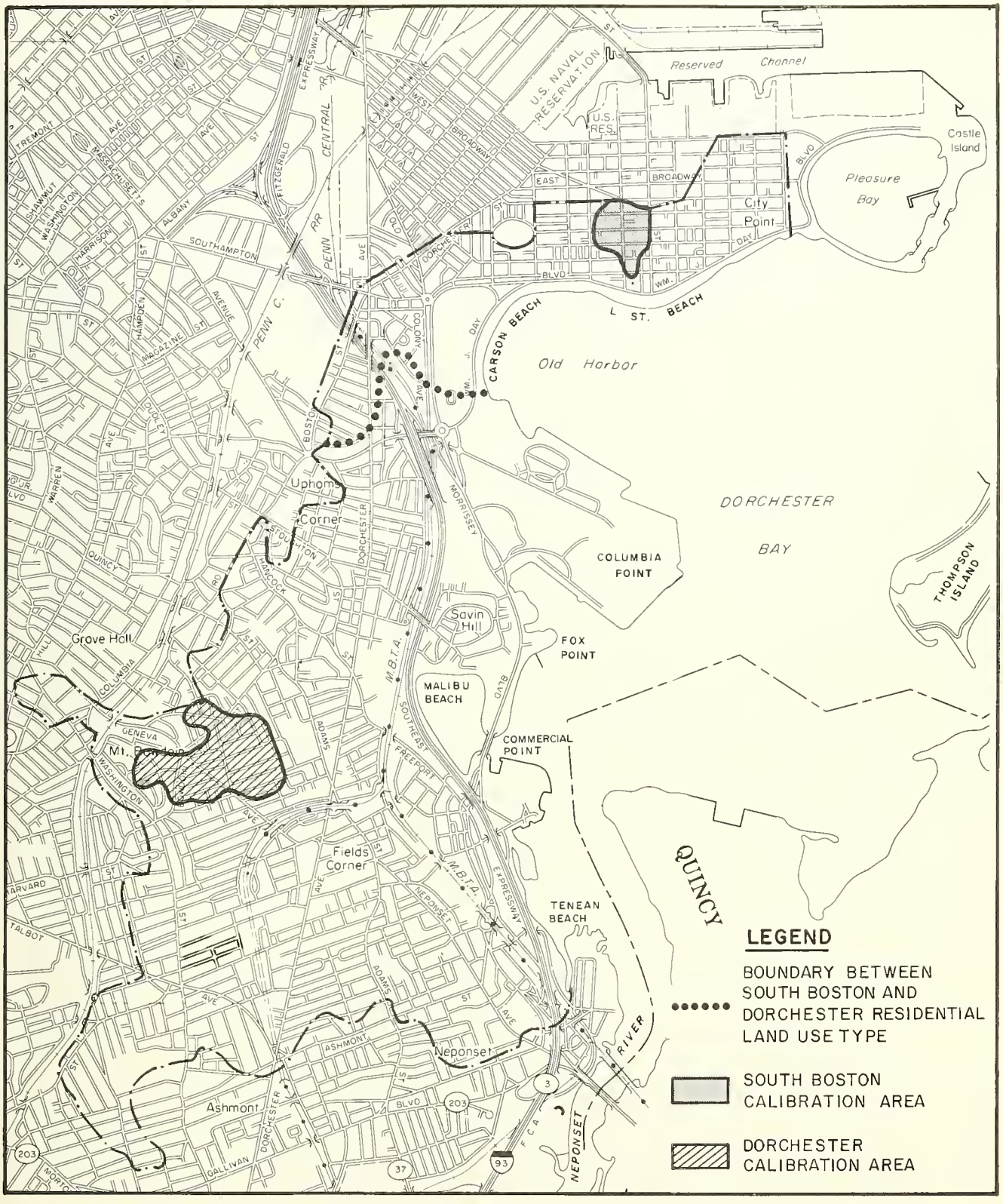


FIG. VII-7 CALIBRATION AREAS FOR RUNOFF MODEL

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areas are considered representative of the two major subareas within the study area, (i.e. South Boston and Dorchester). Field data could not be obtained for additional drainage areas due to the lack of a sufficient number of wet weather events during the field monitoring period. The calibration areas used are appropriate for determining the hydrologic variables associated with residential land use. Characteristics of commercial and grass land uses, such as percent imperviousness, were obtained from literature values which were judged to be applicable to the Dorchester Bay study area. For the area in South Boston, the outflow hydrograph for August 6, 1978, was measured at K and Marine streets; for the area in Dorchester, the outflow hydrograph for October 6, 1978, was measured at Josephine Street. Thus, having input hyetographs and measured outflow hydrographs for each calibration area, it was possible to calibrate the RUNOFF model based on these data.

All of the input parameters required by RUNOFF are summarized in Table VII-1. Many of the necessary values are basin-dependent and are developed from the physical features of the sub-basins of interest. Other data were based on an extensive literature review, and past experience. However, as can be seen from Table VII-1, the values of two important variables resulted from the calibration process. These variables are (1) the percentage of residential land use which is impervious, and (2) the detention (depression) storage to be associated with impervious areas. Detention (or depression) storage represents the amount of rainfall which will be intercepted by land surface irregularities. Overland flow is, therefore, not initiated from a rainfall event until and unless this available storage has been filled.

To arrive at the tabulated values for each of the "calibration" variables, the two areas shown on Figure VII-7 were considered separately. The predominant land use pattern in each of the calibration areas is residential; Table VII-2 summarizes the physical properties of each area. Using this information and the appropriate input hyetographs (see Figures VII-5 and -6), the RUNOFF model was used to simulate the outflow hydrograph with various values of the "unknown" calibration parameters. Figures VII-8 and -9 illustrate the final calibration results which were considered acceptable.

As listed in Table VII-1, the calibration results indicated that .04 inches of detention storage should be associated with impervious areas. However, the percentage of residential land use which is effectively impervious was found to be different for the South Boston and Dorchester areas. In the South Boston area, a value of 50 percent was used to generate Figure VII-8 and a value of 28 percent was used for Figure VII-9 in the Dorchester area. This result reflects the different hydrologic behavior (building type and compositon) of the residential areas which actually exist in these two sections of the study area. Thus, for all overland drainage modeling work performed with both RUNOFF and SEMSTORM, 50 percent of the residential land in South Boston is considered impervious and 28 percent of residential land in Dorchester is considered impervious. Note that these calibration results indicate what is considered "effective" imperviousness. This is not necessarily the same as actual imperviousness (as listed in Table IV-1). In terms of the physical system, for example, the hydrologic connections between roof drains and streets (or backyards) greatly influence the "effective" imperviousness of an area. In order to accurately simulate an area's runoff potential, both RUNOFF and SEMSTORM require information on effective imperviousness rather than actual imperviousness.

TABLE VII-1. RUNOFF INPUT PARAMETERS

| | Value | Source |
|---|---|---|
| **Drainage Basin** | | |
| Area | Basin Dependent | BWSC Plans & USGS Topographical Maps |
| Runoff Length | Basin Dependent | USGS Topographical Maps |
| Runoff Slope | Basin Dependent | USGS Topographical Maps |
| Land Use Composition | Basin Dependent | MAPC Land Use Maps |
| **Impervious - Percent** | | |
| Residential - South Boston | 28 | Calibration[1] |
| Residential - Dorchester | 50 | Calibration[1] |
| Commercial | 95 | Judgment/Literature |
| Grass | 10 | Judgment/Literature |
| **Infiltration[2]** | | |
| Maximum Rate | 1.5 in/hr | Literature-RUNOFF Documentation |
| Minimum Rate | 0.6 in/hr | Literature-RUNOFF Documentation |
| Maximum Amount | 1.0 in | Literature-RUNOFF Documentation |
| **Detention Storage** | | |
| Impervious Areas | .04 in | Calibration[1] |
| Pervious Areas | .25 in | Judgment/Literature |
| **Manning n** | | |
| Impervious Areas | .13 | Literature-RUNOFF Documentation |
| Pervious Areas | .30 | Literature-RUNOFF Documentation |
| **Conduits** | | |
| Size | Conduit Dependent | BWSC Plans & Field Measurements |
| Length | Conduit Dependent | BWSC Plans & Field Measurements |
| Slope | Conduit Dependent | BWSC Plans & Field Measurements |
| Roughness | Conduit Dependent | BWSC Plans & Field Measurements |
| Dry Weather Flow | Variable | Water Consumption Data, Population Data, & Field Measurements |

BWSC Boston Water and Sewer Commission
MAPC Metropolitan Area Planning Council
USGS United States Geographical Survey
[1]See Text.
[2]Infiltration on pervious areas; effectively lost from the pipe system.

Table VII-2. RUNOFF PARAMETERS FOR CALIBRATION AREAS

| LOCATION OF FLOW MEASUREMENT: | K & Marine Streets (South Boston) | Josephine Street (Dorchester) |
|---|---|---|
| DATE OF STORM: | August 6, 1978 | October 6, 1978 |
| Drainage Basin | | |
| Area (acres) | 29 | 94 |
| Land Use Residential | 100% | 100% |
| Runoff Length (ft) | 600 | 1200 |
| Runoff Slope (ft/ft) | .015 | .044 |
| Conduit | | |
| Diameter (ft) | 2.02[1] | 4.50 |
| Length (ft) | 1270 | 800 |
| Slope (ft/mile) | 58.62 | 92.40 |
| Manning's N | .017 | .017 |
| Dry Weather Flow[3] (cfs) | 0.34 | 0.[2] |

(1) This is equivalent to the actual 20 by 30 inch egg-shaped conduit.

(2) Separate storm sewer.

(3) See Chapter V for description of dry weather flow estimates.

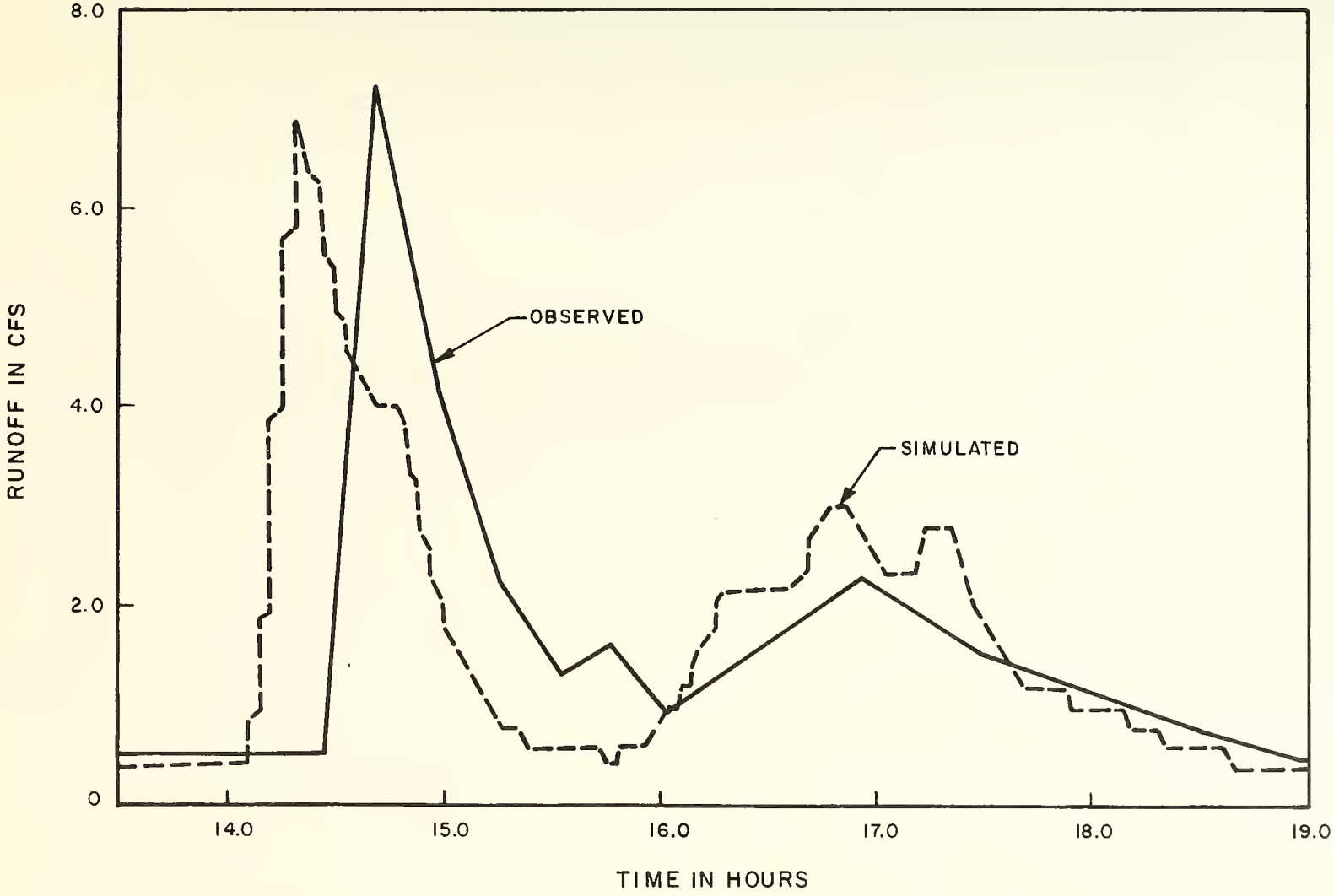


FIG. VII-8 SIMULATED AND OBSERVED HYDROGRAPH FOR THE STORM OF AUGUST 6, 1978 AT K AND MARINE STREETS, SOUTH BOSTON

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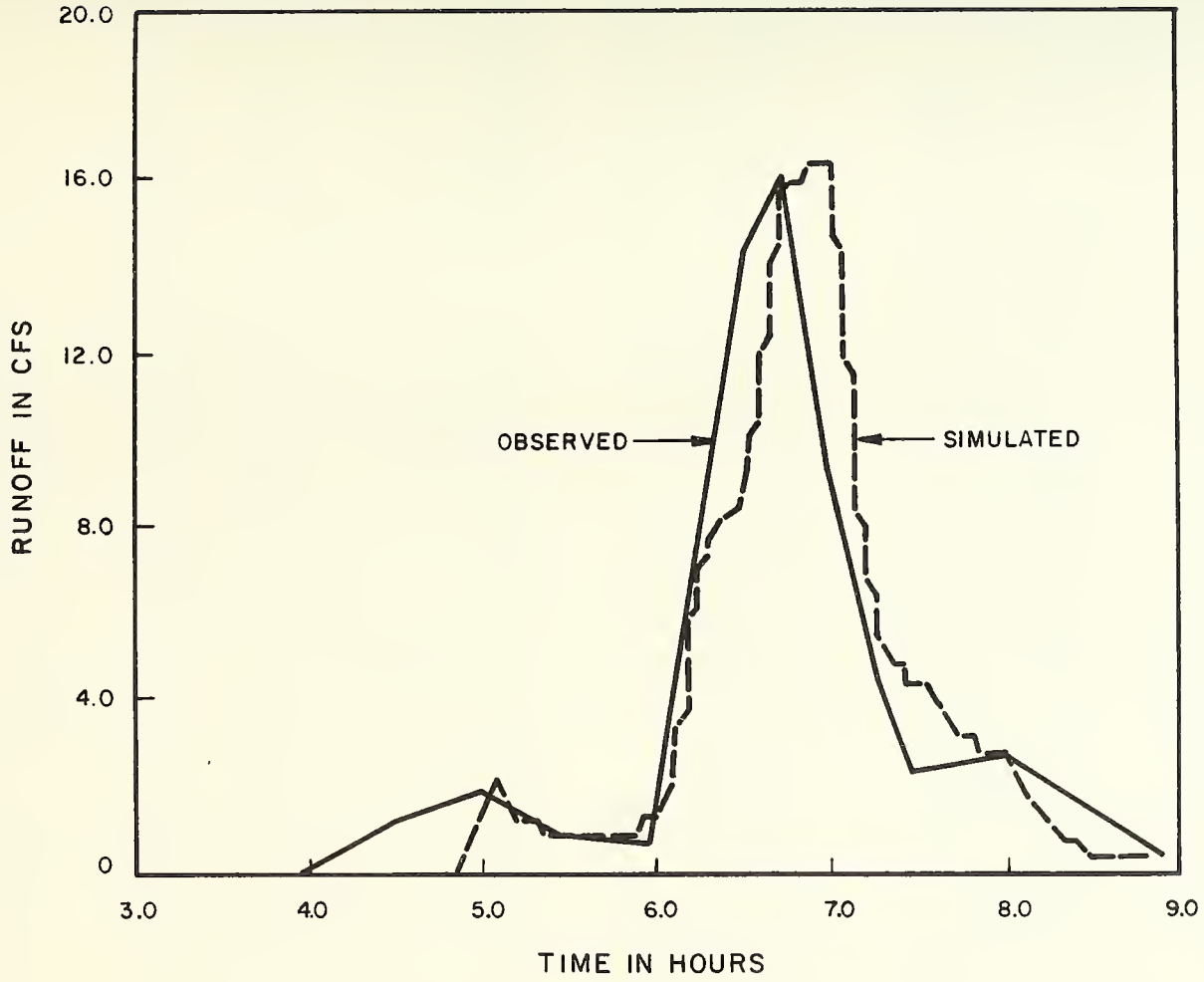


FIG. VII-9 SIMULATED AND OBSERVED HYDROGRAPH FOR THE STORM OF OCTOBER 6, 1978 AT JOSEPHINE STREET, DORCHESTER

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Use of Model. As illustrated on Figure VII-3, the RUNOFF model was used to help determine various hydrologic calibration parameters for the SEMSTORM modeling work. This will be explained further in the following section. RUNOFF itself, however, had a significant role in the overall combined sewer modeling work. As will be described in the section concerning TRANSPORT, inflow hydrographs for 53 separate sub-areas were generated with RUNOFF and subsequently used as input to the TRANSPORT network. Details of this work are presented later in this chapter.

## The SEMSTORM Model

SEMSTORM is a planning level program that simulates the flows and amounts of pollutants generated from a combined sewer area. The model was developed for the evaluation of pollutant abatement practices from point and nonpoint sources of pollutants. The model was originally applied to the South Eastern Michigan area and has been applied recently to many other areas. For the purpose of limiting the following discussion, only the SEMSTORM capabilities that were used in this CSO study are presented here.

Operation. The SEMSTORM model focuses attention on the "lumped" CSO effect of rainfall and runoff on rather large drainage areas. The analysis begins with the delineation of those areas that hydrogically contribute to particular CSO outfalls. The program continuously simulates overflow quantity and quality generated by these areas, using rainfall-runoff and land-use pollutant relationships along with a long-term record of hourly precipitation.

Figure VII-10 shows elements of the physical system used in the SEMSTORM conceptualization. No detailed dynamic simulation of the performance of the drainage network is attempted with SEMSTORM. However, this simplification is offset by the long-term hourly simulation of the system over several years. The output concerning the overflow time series produces estimates of the annual frequency and magnitudes of overflow occurrences.

Direct runoff from precipitation is computed using a modified form of the familiar rational formula. The variation of the runoff characteristics from one area to another is reflected in the selection of a number of land-use categories. Runoff coefficients selected for each land-use within an area are "area-weighted" to produce a single runoff coefficient for each SEMSTORM area; however, runoff is assumed to take place only after available depression storage is filled. Depression storage is emptied by evaporation at a rate which is variable by month.

Pollutants associated with overland flow are directly related to individual land use types. The amount of organic material collected by overland runoff, expressed in terms of $BOD_5$, is considered to be proportional to the Total Suspended Solids (TSS) removed. Accordingly, only TSS response to rainfall and runoff is simulated. TSS are postulated to accumulate on land surface at a diminishing rate as the dry period lengthens after a runoff-producing rainfall. At the next runoff-producing rainfall, TSS which have accumulated are removed as a function of the runoff intensity.

PHYSICAL SYSTEM

"SEMSTORM" CONCEPTUALIZATION

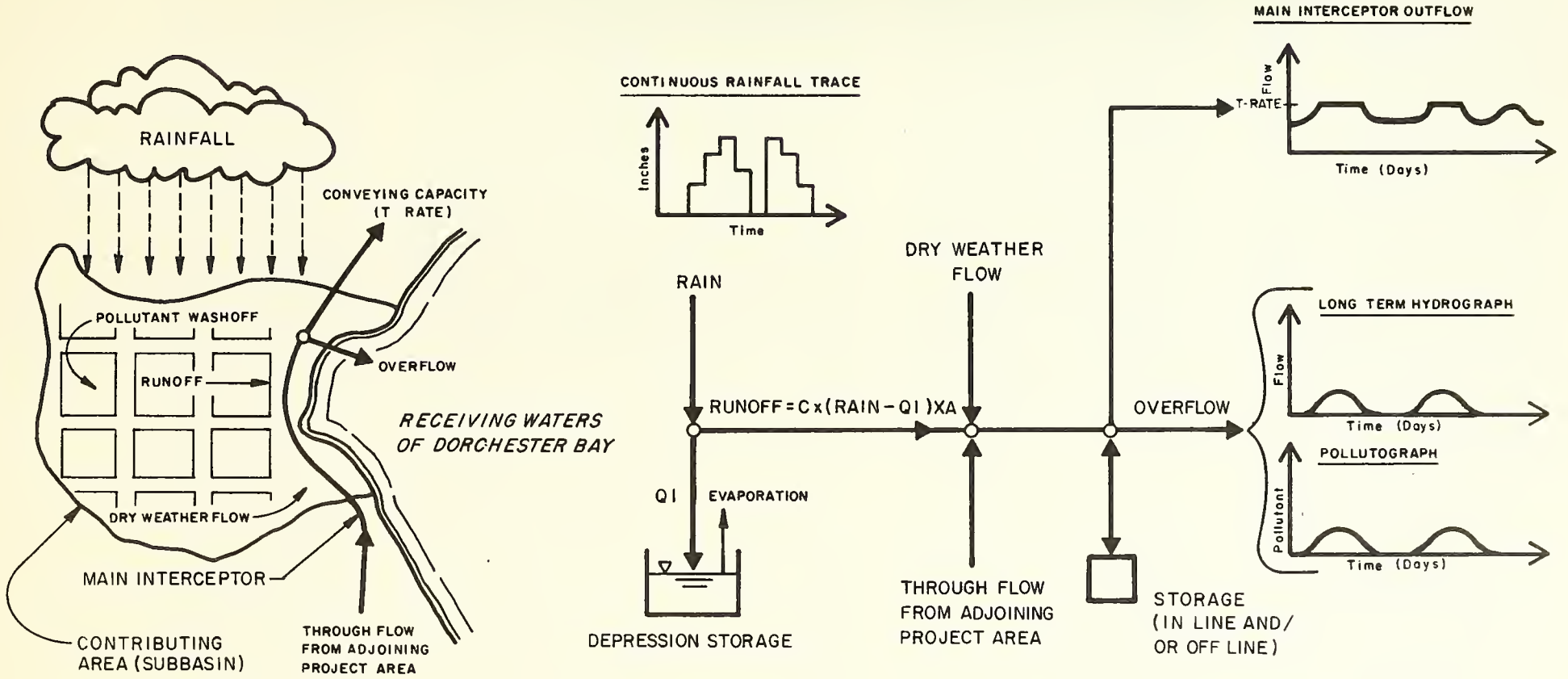


FIG. VII-10 CONCEPTUALIZATION OF URBAN HYDROLOGY IN SEMSTORM

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Simulated runoff and associated pollutant loads are added to dry weather flow generated in the same SEMSTORM sub-basin. The main collector conduit that receives the flow provides a dual capability for conveyance of the total flow, as well as for in-line storage of some volume. Exceedance of both capacities (conveyance and storage) results in an overflow event. SEMSTORM retains information concerning these events and develops annual statistics regarding the overflows.

The above model description is purposely general. For more detail on conceptual or computational features of the SEMSTORM model, refer to the model documentation (see References). The remainder of this section presents the detail of the use of SEMSTORM in the Dorchester Bay area.

Use of Model. Figure VII-11 illustrates how the Dorchester Bay area was subdivided into seven areas for the purpose of SEMSTORM modeling. Also shown on the figure are the "separate sewer" areas which were also analyzed with SEMSTORM. Areas immediately adjacent to Dorchester Bay drain directly to the receiving waters and were not modeled in the study. The seven combined sewer sub-areas, however, were used to assess the long-term CSO statistics associated with the existing conveyance and storage capacities in the sewer. For the evaluation of future conditions, only three SEMSTORM combined sewer areas were used to simplify the screening process. These three areas--designated A1, A2, and A3--are shown on Figure VII-12. Allocation of SEMSTORM overflow statistics to several overflows in a single area was based on area-weighting the model results.

Data associated with all of the SEMSTORM sub-areas are given in Table VII-3. The percent of residential land which is considered effectively impervious for each area coincides with the RUNOFF calibration results and whether the area is in Dorchester (28%) or South Boston (50%). The impervious percentages associated with other land use categories were determined on a case-by-case basis from land use maps.

Climatological Data. For every SEMSTORM simulation, almost 29 years of hourly input precipitation data were used to "drive" the model. The period of record was from May 1948 to December 1976. The rain gage selected was that located at Logan Airport in Boston. Monthly evaporation rates used (to empty depression storage) are given in Table VII-4.

A uniform areal distribution of both rainfall and evaporation was retained for this analysis. This is justifiable because of the relatively small size of the study area. The origin and type of precipitation event is not assumed to affect the overall statistics of the simulated overflows. This is one of the advantages of the long-term SEMSTORM simulations.

Runoff Characteristics. For the purpose of SEMSTORM modeling, the runoff characteristics of an area are lumped into a runoff coefficient C. The runoff coefficient is based on the degree of imperviousness in the sub-area of interest. The runoff characteristics of a given land use category, in turn, are based on the mix of pervious and impervious land in that category. The functional relationship used between percent impervious and the runoff coefficient for the sub-areas is shown on Figure VII-13. The relationship was developed from data and measurements made

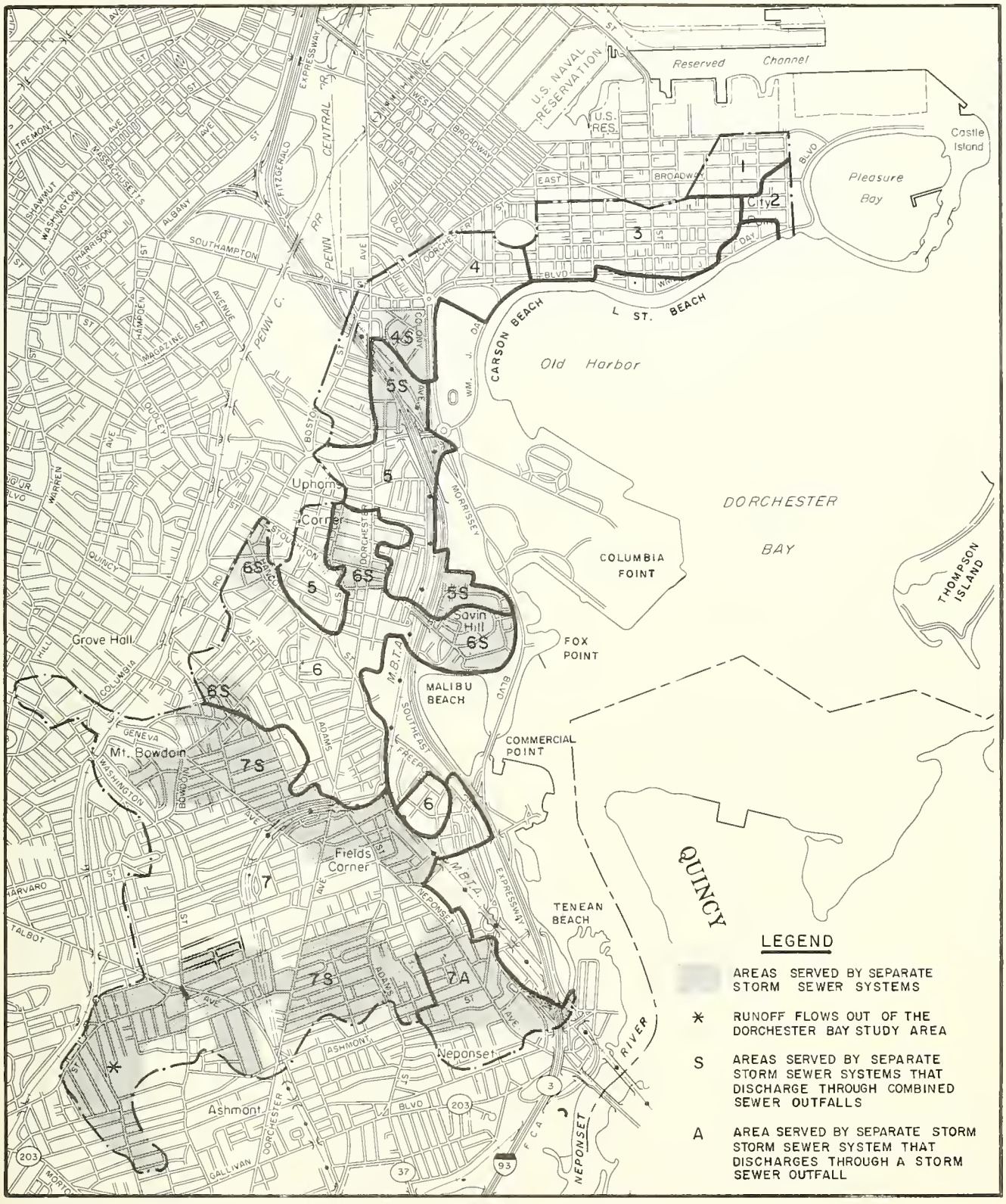


FIG. VII-11 SEMSTORM SUBBASINS USED FOR ASSESSMENT OF EXISTING CONDITIONS

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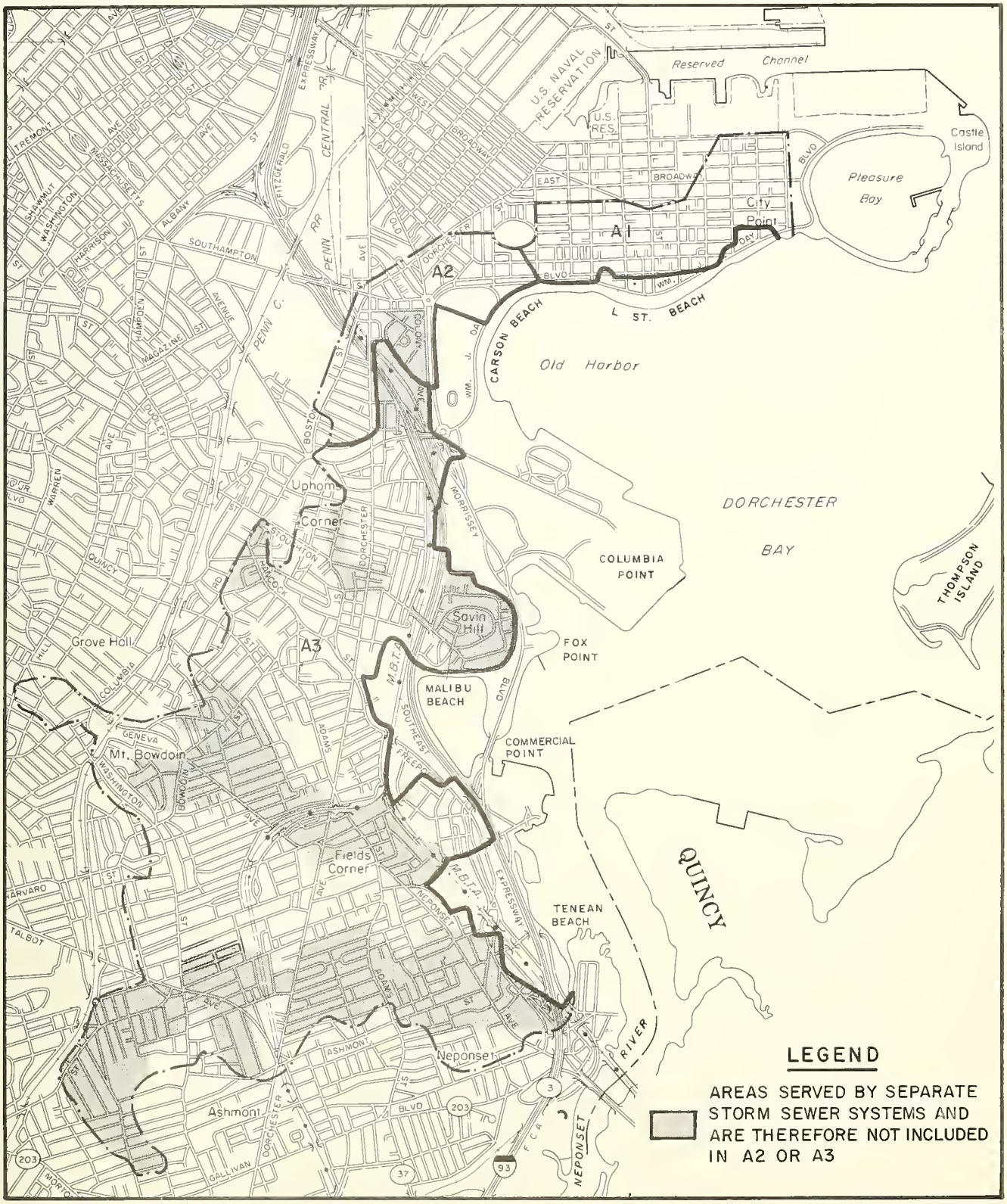


FIG. VII-12 AGGRAGRATED SEMSTORM SUBBASINS USED FOR ASSESSMENT OF FUTURE CONDITIONS

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Table VII-3. SEMSTORM HYDROLOGIC PARAMETERS

| Subbasin | Area (acres) | Land Use (%) | | | Percent Impervious | | | Depression[4]* Storage (inches) | Runoff Coefficient (Cq) | Dry Weather Flow (mgd) | Existing[5] Storage (mg) | Existing[5] Conveyance Capacity (mgd) |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | Residential | Commercial | Open | Residential* | Commercial | Open | | | | | |
| 1 | 47 | 57.2 | 32.1 | 10.7 | 50 | 95 | 10 | .124 | .501 | .23 | .0049 | .33 |
| 2 | 31 | 93. | 0. | 7 | 50 | 90 | 10 | .151 | .407 | .20 | .0096 | .25 |
| 3 | 142 | 89.7 | 8.8 | 1.5 | 50 | 95 | 10 | .138 | .445 | 1.06 | .15 | 1.3 |
| 4 | 124 | 73.2 | 25.1 | 1.7 | 50 | 95 | 10 | .123 | .496 | 1.03 | .31 | 1.3 |
| 5 | 174 | 77.9 | 21.1 | 1.0 | 28 | 98 | 10 | .157 | .414 | 1.2 | .14 | 5.2 |
| 6 | 310 | 66.7 | 27.1 | 6.2 | 28 | 100 | 10 | .150 | .439 | 2.31 | .082 | 9.8 |
| 7 | 555 | 77.0 | 19.0 | 4.0 | 28 | 95 | 10 | .163 | .398 | 6.05 | .0 | 6.1 |
| 4S[1] | 24 | 100. | 0. | 0. | 50 | - | - | .145 | .420 | - | - | - |
| 5S[1] | 54 | 42.6 | 55.3 | 2.1 | 28 | 98 | 10 | .109 | .561 | - | - | - |
| 6S[1] | 100 | 86.7 | 9.9 | 3.4 | 28 | 100 | 10 | .174 | .365 | - | - | - |
| 7S[1] | 340 | 87.3 | 7.3 | 5.4 | 28 | 95 | 10 | .179 | .342 | - | - | - |
| 7A[2] | 118 | 63.0 | 37.0 | 0.0 | 28 | 95 | - | .136 | .448 | - | - | - |
| A1[3] | 220 | 83.1 | 12.7 | 4.2 | 50 | 95 | 10 | .136 | .452 | 1.49 | .17 | 1.9 |
| A2[3] | 129 | 73.2 | 25.1 | 1.7 | 50 | 95 | 10 | .123 | .496 | 1.03 | .31 | 1.3 |
| A3[3] | 1039 | 74.1 | 74.1 | 4.1 | 28 | 97 | 10 | .161 | .406 | 9.56 | .22 | 21. |

[1]Areas served by separate storm sewers which discharge into CSO outfalls.

[2]Separate storm sewer area which drains to its own outfall.

[3]Aggregated areas for use in 'screening'.

[4]Depression storage based on impervious/pervious composition (.04 inches-impervious; .25 inches pervious).

[5]Existing 'Storage' and 'Conveyance Capacity' are tabulated for reference; SEMSTORM was used for screening with alternative values as well.

*Values obtained from RUNOFF calibration.

Table VII-4. SEMSTORM EVAPORATION RATES[1]

| Month | Evaporation Rate (in/day) |
|---|---|
| January | .051 |
| February | .051 |
| March | .064 |
| April | .069 |
| May | .158 |
| June | .166 |
| July | .214 |
| August | .179 |
| September | .113 |
| October | .087 |
| November | .051 |
| December | .049 |

(1) Data for April, May, June, July, August and September are based on monthly total pan data recorded at the Chestnut Hill Reservoir.

Data for January, February, March, October, November and December are based on representative data for the New England area reported in the Climatic Atlas of the United States (U.S. Department of Commerce, 1974).

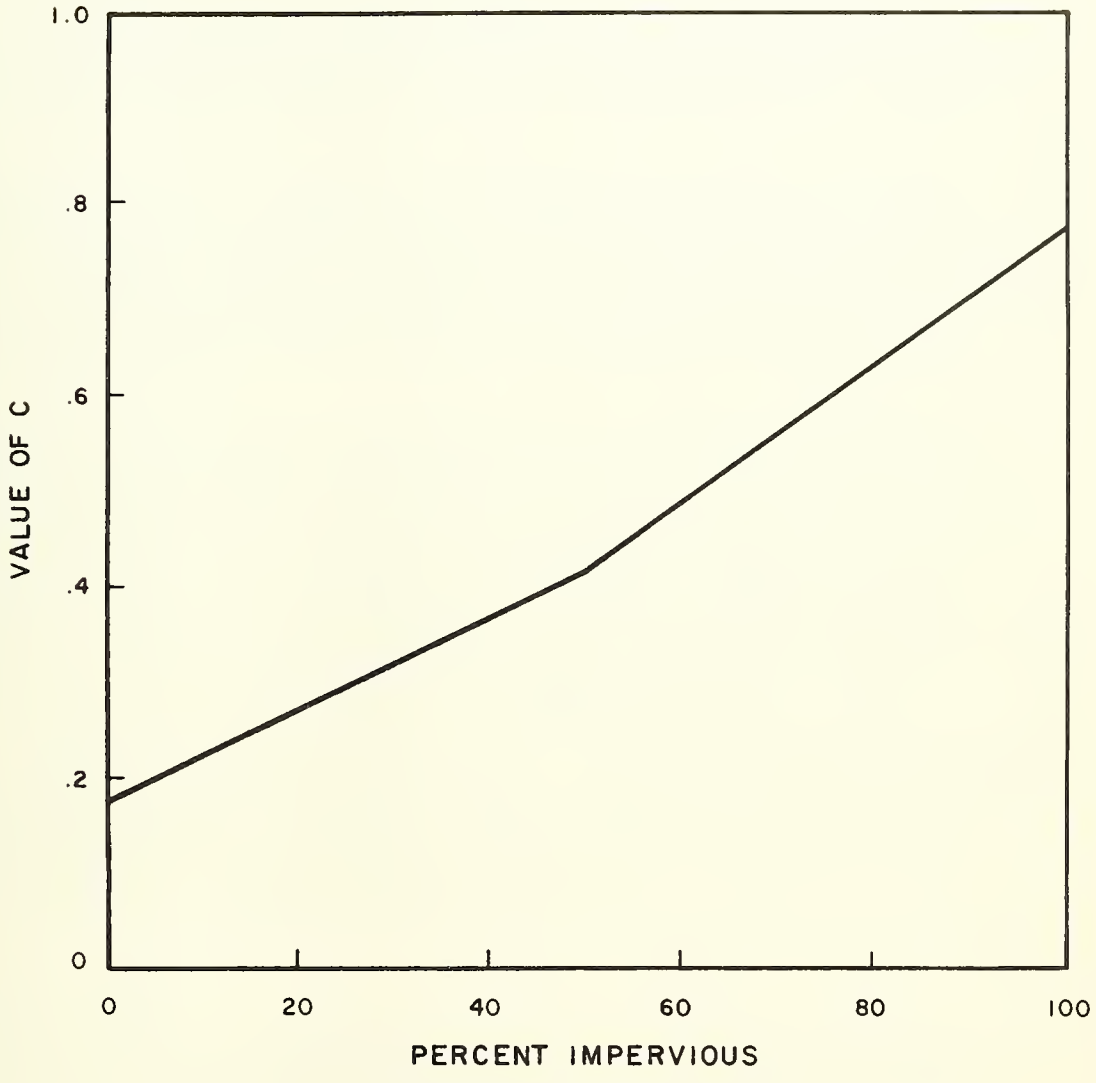


FIG. VII-13 RUNOFF COEFFICIENT AS A FUNCTION OF PERCENT IMPERVIOUSNESS

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during the field program. The C coefficients tabulated for each SEMSTORM area in Table VII-3 are based on the total imperviousness for the area and the relationship given on Figure VII-13.

Quality Parameters. The quality parameters used in the SEMSTORM simulations are grouped and presented in Table VII-5. Three water quality parameters were considered: total suspended solids (TSS), biochemical oxygen demand ($BOD_5$), and total coliform. The concentrations of these constituents in the dry weather flow were determined by field sampling, as discussed in Chapter V.

As mentioned previously, SEMSTORM bases runoff quality simulation on TSS. Each mass unit of TSS, in turn, is associated with a certain mass of $BOD_5$. The representative ratio shown in Table VII-5 (0.4125) was based on field monitoring samples of the washoff in separate sewers. The field sampling also indicated a total coliform concentration in the runoff of $2.16 \times 10^6$ coliforms per 100 ml and was, therefore, also incorporated in the SEMSTORM model.

The rate of accumulation of TSS between rainfall events was developed from previous experience in similar studies in the Detroit area, and is taken to decrease with time, up to a maximum accumulation of 41 lb/acre for the residential land use category, 74 lb/acre for commercial land use, and 10 lb/acre for open space. The parameter characterizing the decrease in the rate of TSS accumulation is known as the "half saturation" constant and was set at 1.0 day for this study (see Table VII-5). Also, the pollutant washoff exponent was fixed at a value of 2.0 in units consistent with the units of SEMSTORM (see Table VII-5).

Output. Based on the data presented above, SEMSTORM was used to simulate the performance of the existing and future system alternatives. The results of each simulation exercise were--for each overflow outfall--a set of exceedance curves for the volume, TSS, $BOD_5$, and Total Coliform in the overflow, as shown on Figure VII-14.

## The TRANSPORT Model

As discussed earlier, the TRANSPORT model was used in the development of the Dorchester Bay area CSO facilities plan to simulate the detailed hydraulic response of the sewer system for selected rainfall events during detailed evaluation of selected alternatives. How and why particular rainfall events were selected is presented in the last section of this chapter. For the moment, however, the text will focus only on the operation and use of the TRANSPORT model itself.

Operation. A schematic representation of the input and output associated with the TRANSPORT model's operation is shown on Figure VII-15. The model network is composed of links (conduits) and nodes (junctions) for which physical and operational data are supplied. The link-node description facilitates both the discrete representation of the prototype sewer system and the mathematical solution of the gradually-varied, unsteady flow equations which are the computational basis of the model.

Table VII-5. SEMSTORM QUALITY PARAMETERS

| | |
|---|---|
| Dry Weather Flow | |
| TSS | 130 mg/l |
| $BOD_5$ | 165 mg/l |
| Total Coliform | 23 x $10^6$ coliforms/100 ml |
| TSS-$BOD_5$ Conversion Factor[1] | 0.4125 |
| Total Coliform Concentration in Runoff | 2.16 x $10^6$ coliforms/100 ml |
| TSS Maximum Accumulation[2] | |
| Residential | 41 pounds/acre |
| Commercial | 74 pounds/acre |
| Open | 10 pounds/acre |
| Half-Saturation Constant[2] | 1.0 days |
| Pollutant Washoff Exponent[3] | 2.0 |

---

[1] Mass of $BOD_5$ per unit mass of TSS.

[2] $AC_i = TSS_i^{max} \left( \frac{HAFSAT}{(HAFSAT+\Delta t)} 2 \right)$

Where $AC_i$ = TSS accumulation for land use i (lb/acre/day)

$TSS_i^{max}$ = TSS maximum accumulation for land use i (lb/acre)

$\Delta t$ = Time since runoff (days)

HAFSAT = Half-saturation constant (days) which is a rate factor specifying how rapidly the rate of TSS accumulation decreases with time since last runoff.

[3] $M_{it} = P_{it} (1-e^{-KR\Delta t}) / \Delta t$

Where $M_{it}$ = Mass rate of TSS removed from land use i during $\Delta t$

$P_{it}$ = TSS load on land use i at time t

R = Rate of runoff (in/hr) during $\Delta t$

K = Pollutant washoff exponent.

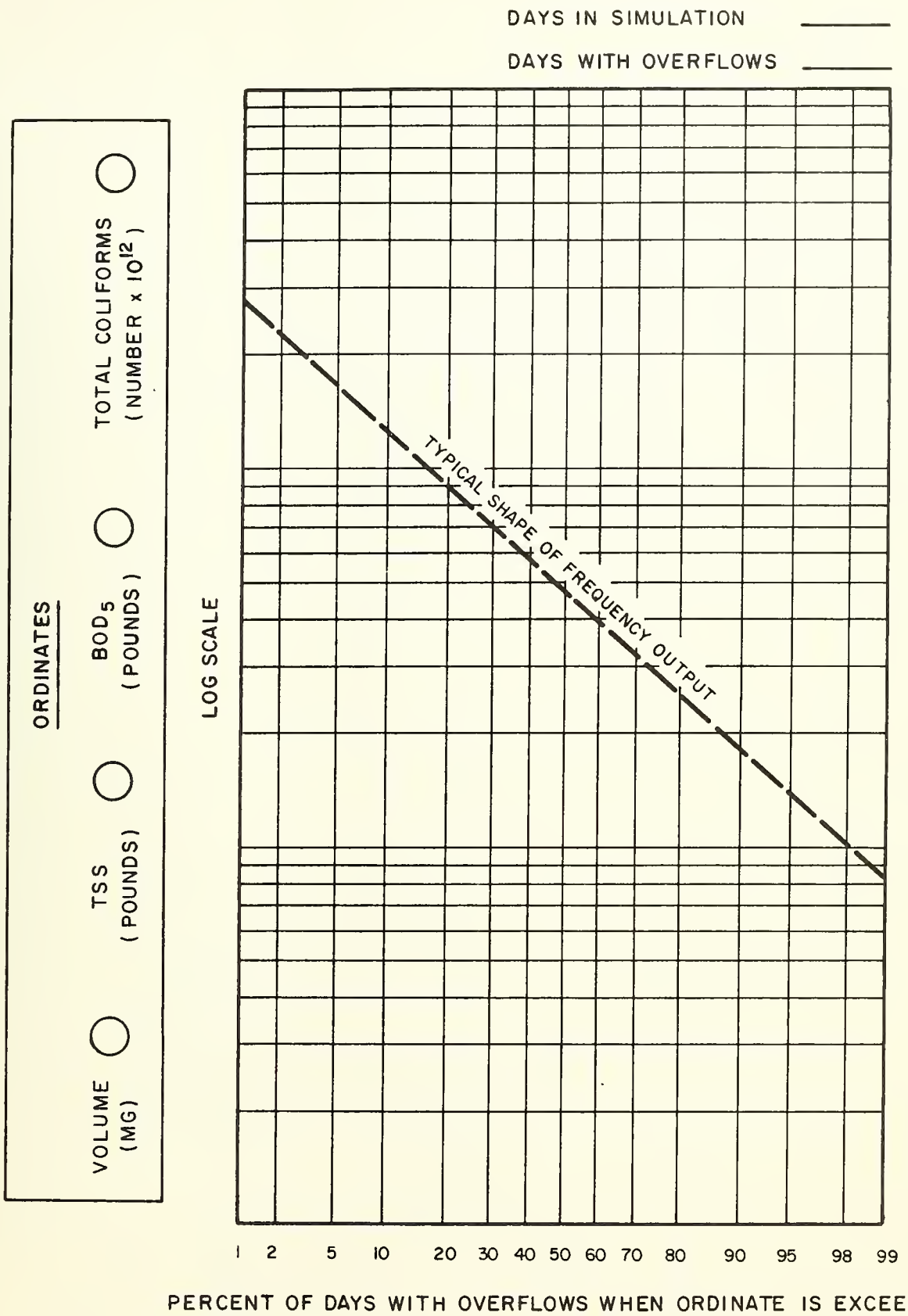
DAYS IN SIMULATION
DAYS WITH OVERFLOWS
ORDINATES
VOLUME (MG)
TSS (POUNDS)
BOD5 (POUNDS)
TOTAL COLIFORMS (NUMBER x 10^12)
LOG SCALE
TYPICAL SHAPE OF FREQUENCY OUTPUT
1 2 5 10 20 30 40 50 60 70 80 90 95 98 99
PERCENT OF DAYS WITH OVERFLOWS WHEN ORDINATE IS EXCEEDED

FIG. VII-14 MANNER IN WHICH SEMSTORM RESULTS ARE DISPLAYED

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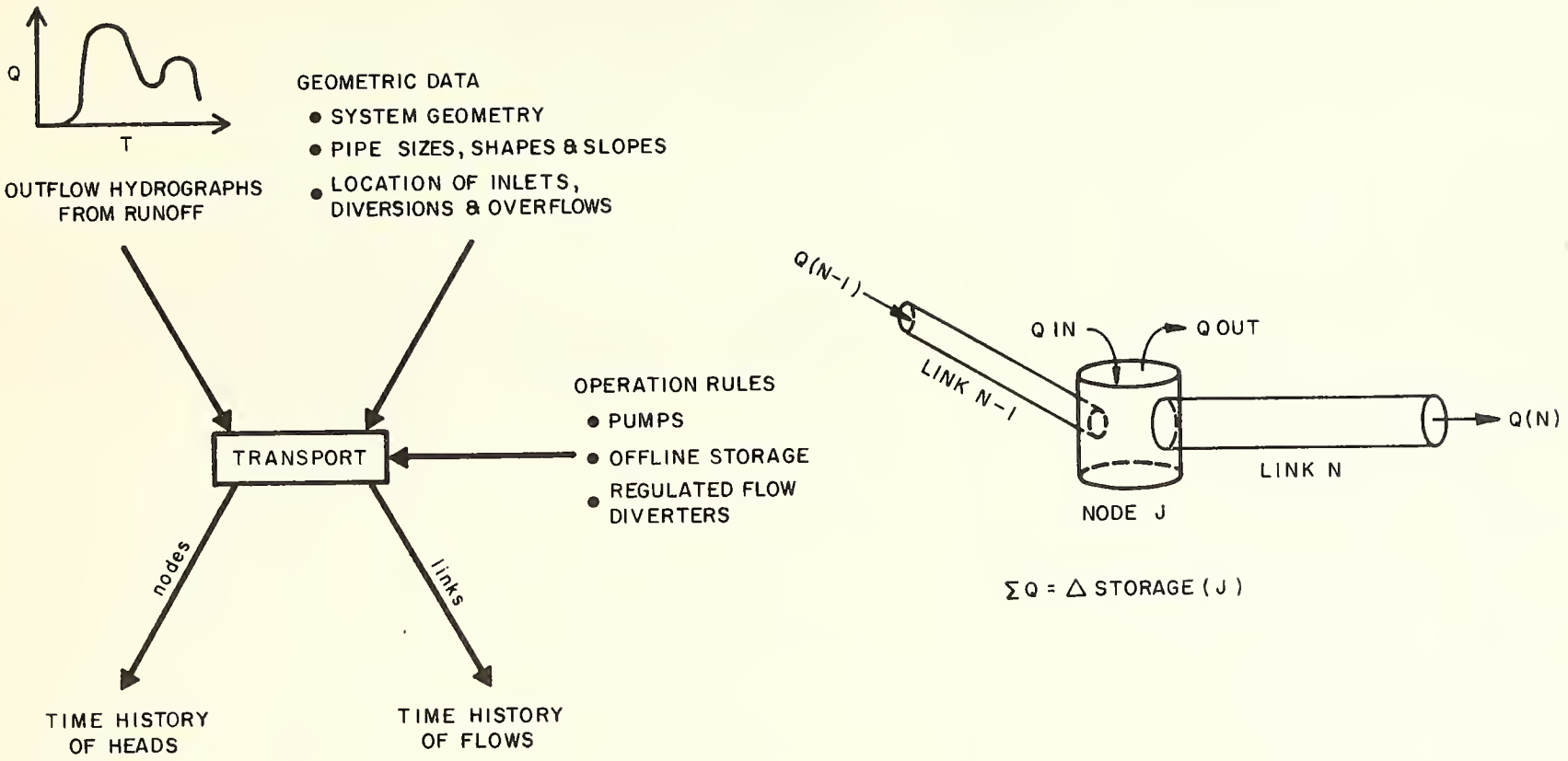


FIG. VII-15 REPRESENTATION OF THE TRANSPORT MODEL LINK-NODE CONCEPTUALIZATION

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Links transmit flow from node to node. Properties associated with the links are internal roughness coefficients, length, cross-sectional area, hydraulic radius and surface width. The last three properties are functions of the instantaneous depth of flow. The primary dependent variable in the links is the discharge, Q. It is assumed that Q is constant in the link, while velocity and the cross-sectional area of flow, or depth, are variable in the link over each modeling time increment.

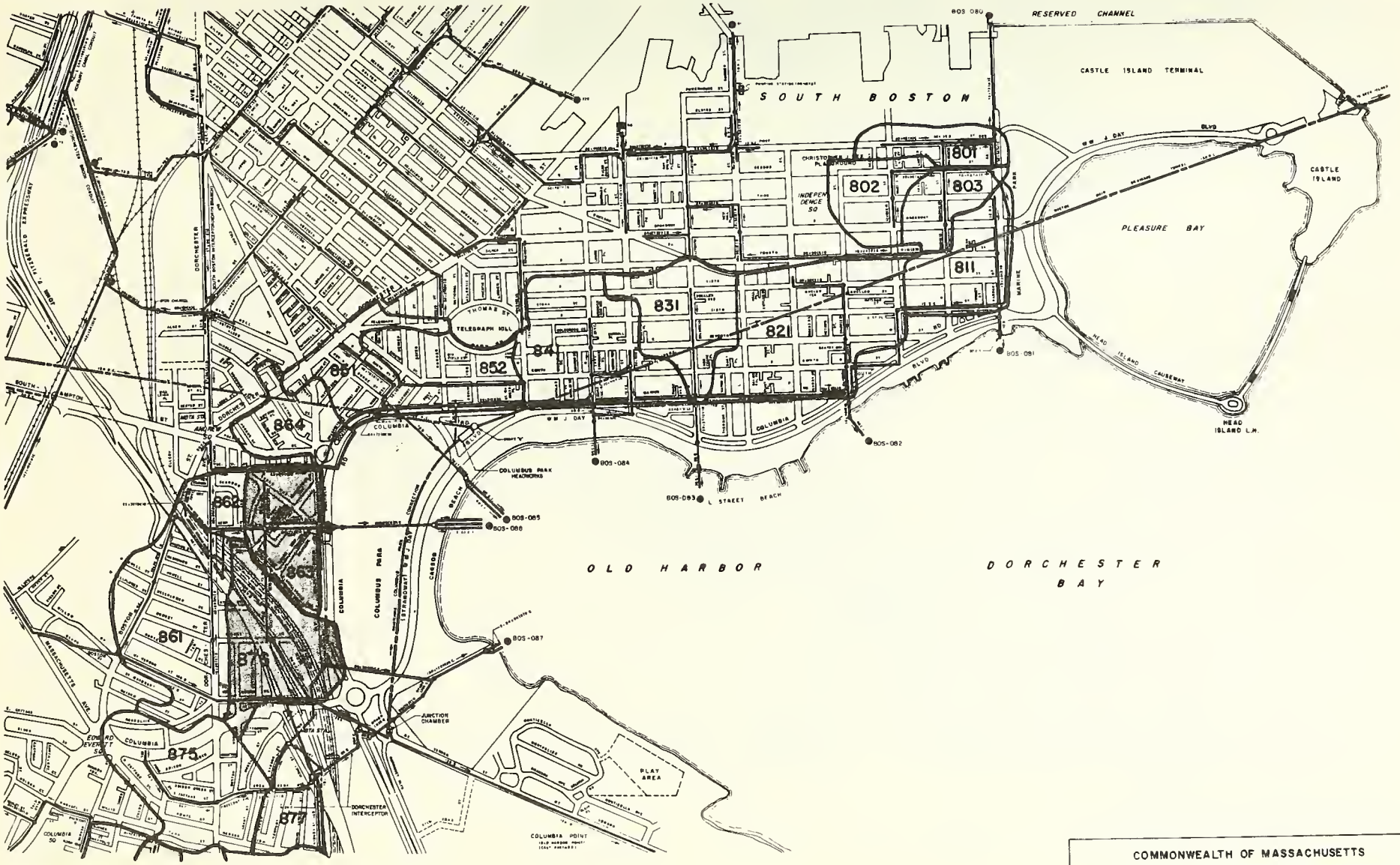
Nodes are the storage elements of the system and correspond to manholes or pipe junctions in the physical system. The variables associated with a node are volume, height (head), and surface area. The primary dependent variable is the head, H, which is assumed to be changing in time but constant throughout any one node. Inflows, such as inlet hydrographs, and outflows, such as diversions, take place at the nodes of the idealized sewer system network. The volume of a node at any time is equivalent to the water volume in one-half of the length of pipes connected to that node. The change in nodal volume during a given modeling time step forms the basis of head and discharge calculations.

For precise mathematical details associated with operation of the TRANSPORT model, the reader is referred to the program documentation. Note, however, that since TRANSPORT merely solves dynamic flow equations for the sewer system, if the actual system is properly depicted as a network, the TRANSPORT output will be representative of what would actually occur.

Application to Dorchester Bay Area. In order to apply TRANSPORT to the study area, it was first necessary to delineate the appropriate RUNOFF sub-areas that provide input hydrographs to TRANSPORT. Figure VII-16 (A, B, and C) illustrates the 53 RUNOFF sub-areas which were used in this modeling effort. Table VII-6 (A and B) summarizes the physical properties associated with each area.

As described in the earlier discussion of the RUNOFF model, the overland flow portion of the output from the RUNOFF sub-areas is routed through conduits before being input to TRANSPORT. Due to the kinematic wave approximation used in RUNOFF, these routing conduits cannot be backwater effected under the constraint that the friction slope must equal the pipe slope. Thus, the RUNOFF conduits used represent the upstream portions of the sewer system.

Figure VII-17 (A and B) illustrates the TRANSPORT network which was established for the current configuration of the Dorchester Bay area's sewer system. Also shown (as dashed lines) are the RUNOFF conduits which provide input hydrographs to TRANSPORT at the locations shown. Table VII-7 (A and B) summarizes the data associated with the RUNOFF conduits. Appendix A presents the data associated with the TRANSPORT nodes and links as they are numbered in the figure.




AREAS SERVED BY SEPERATE STORM SYSTEMS

FIG. VII-16A RUNOFF SUBBASINS

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AREAS SERVED BY SEPERATE STORM SYSTEMS

FIG. VII-16 B RUNOFF SUBBASINS

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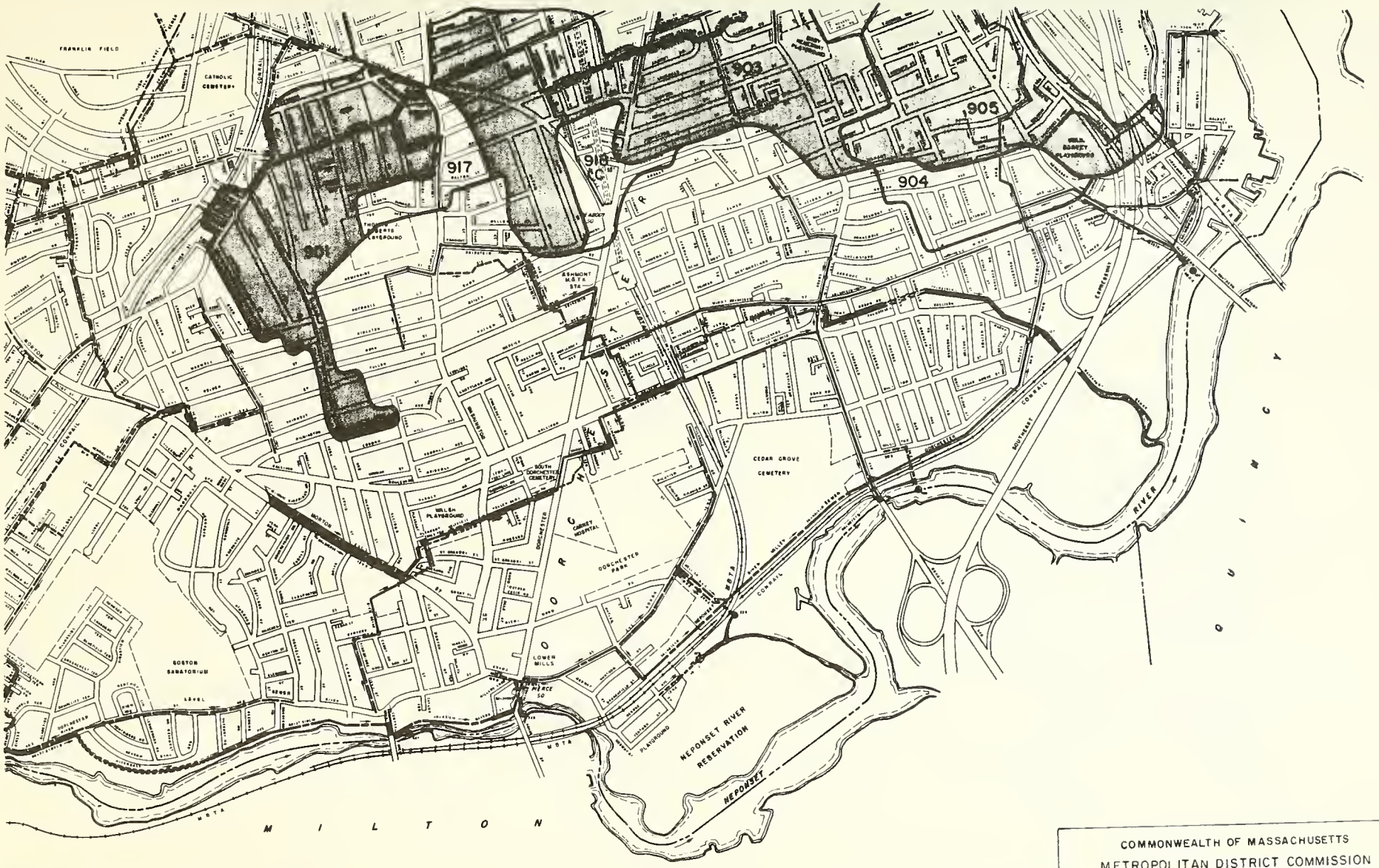


AREAS SERVED BY SEPERATE STORM SYSTEMS

FIG. VII-16C RUNOFF SUBBASINS

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DORCHESTER BAY AREA FACILITIES PLAN

Table VII-6A. AREA DATA FOR RUNOFF BLOCK - SOUTH BOSTON[1]

| Subarea | Conduit[2] | Area (acres) | Runoff Length (feet) | Runoff Slope (ft/ft) | Land Use (%) | | |
|---|---|---|---|---|---|---|---|
| | | | | | Residential | Commercial | Grass |
| 801 | 1080 | 10 | 200 | .005 | 10 | 90 | 0 |
| 802 | 2080 | 19 | 525 | .040 | 68 | 16 | 16 |
| 803 | 3080 | 18 | 550 | .012 | 72 | 17 | 11 |
| 811 | 1081 | 27 | 700 | .017 | 93 | 0 | 7 |
| 821 | 1082 | 72 | 1100 | .001 | 89 | 11 | 0 |
| 831 | 1083 | 29 | 600 | .015 | 97 | 3 | 0 |
| 841 | 1084 | 43 | 650 | .032 | 86 | 9 | 5 |
| 851 | 1085 | 21 | 1100 | .014 | 100 | 0 | 0 |
| 852 | 2085 | 26 | 500 | .058 | 85 | 15 | 0 |
| 861 | 2487 | 61 | 500 | .005 | 62 | 38 | 0 |
| 862 | 2287 | 8 | 250 | .008 | 68.5 | 31.5 | 0 |
| 863 | 2587 | 24 | 600 | .003 | 100 | 0 | 0 |
| 864 | 2387 | 20 | 650 | .008 | 66 | 22 | 12 |
| 879 | 2987 | 55 | 1150 | .001 | 72.7 | 0 | 27.3 |

[1] See Figure VII-16 for location of subareas.

[2] Conduit that collects the surface drainage and dry weather flow from the subarea.

Table VII-6B. AREA DATA FOR RUNOFF BLOCK - DORCHESTER[1]

| Subarea | Conduit[2] | Area (acres) | Runoff Length (feet) | Runoff Slope (ft/ft) | Land Use (%) | | |
|---|---|---|---|---|---|---|---|
| | | | | | Residential | Commercial | Grass |
| 808 | 8189[3] | 27 | 400 | .080 | 71.5 | 28.5 | 0 |
| 809 | 89105[3] | 9 | 600 | .050 | 85 | 10 | 5 |
| 871 | 1087 | 21 | 700 | .133 | 87 | 13 | 0 |
| 872 | 6087 | 39 | 600 | .020 | 79 | 21 | 0 |
| 873 | 3087 | 21 | 650 | .016 | 99 | 1 | 0 |
| 874 | 4087 | 26 | 1200 | .058 | 85 | 15 | 0 |
| 875 | 5087 | 46 | 750 | .004 | 69 | 30 | 1 |
| 876 | 7887 | 33 | 1000 | .010 | 14 | 82.5 | 3.5 |
| 877 | 7087 | 25 | 800 | .011 | 91 | 9 | 0 |
| 881 | 1088 | 25 | 400 | .040 | 58 | 33 | 9 |
| 882 | 3088 | 5 | 600 | .008 | 63.5 | 36.5 | 0 |
| 883 | 5088 | 16 | 800 | .010 | 96 | 4 | 0 |
| 884 | 2088 | 14 | 1200 | .002 | 85 | 15 | 0 |
| 885 | 4088 | 11 | 200 | .010 | 83.6 | 16.4 | 0 |
| 888 | 9088 | 24 | 500 | .006 | 33 | 67 | 0 |
| 890 | 1189 | 24 | 1700 | .041 | 47.5 | 32.5 | 20 |
| 891 | 1089 | 26 | 1450 | .001 | 74 | 19 | 7 |
| 892 | 9089 | 35 | 2400 | .006 | 60 | 35 | 5 |
| 893 | 8089 | 14 | 500 | .020 | 91.6 | 8.4 | 0 |
| 894 | 5089 | 53 | 1050 | .027 | 85 | 10 | 5 |
| 895 | 6089 | 20 | 500 | .030 | 78.6 | 21.4 | 0 |
| 896 | 7089 | 32 | 825 | .055 | 96.5 | 3.5 | 0 |
| 897 | 3089 | 28 | 1250 | .033 | 47.5 | 32.5 | 20 |
| 898 | 4089 | 22 | 1200 | .033 | 58 | 42 | 0 |
| 899 | 6088 | 40 | 800 | .050 | 90 | 1.5 | 8.5 |
| 902 | 2090 | 47 | 1900 | .011 | 72 | 27 | 0 |
| 903 | 4090 | 180 | 2000 | .030 | 93 | 3.5 | 3.5 |
| 904 | 5090 | 22 | 2450 | .018 | 95 | 5 | 0 |
| 905 | 6090[3] | 118 | 1000 | .020 | 63 | 37 | 0 |
| 906 | 9101[3] | 53 | 750 | .011 | 83.5 | 14 | 2.5 |
| 907 | 8090 | 43 | 1750 | .023 | 90.5 | 9.5 | 0 |
| 909 | 1290 | 38 | 1200 | .024 | 73.6 | 25 | 1.4 |
| 910 | 1490[3] | 94 | 1200 | .044 | 96 | 4 | 0 |
| 911 | 9112[3] | 70 | 1000 | .019 | 80.5 | 10.5 | 9 |
| 912 | 1990 | 41 | 1400 | .022 | 85 | 7 | 8 |
| 913 | 1190 | 55 | 1900 | .024 | 84.5 | 15.5 | 0 |
| 914 | 8290 | 34 | 1200 | .004 | 57.5 | 38 | 4.5 |
| 915 | 8390 | 32 | 900 | .011 | 61.5 | 6.5 | 32 |
| 916 | 9090 | 14 | 800 | .001 | 50 | 50 | 0 |
| 917 | 1090 | 22 | 450 | .016 | 40.6 | 37.4 | 22 |
| 918 | 3090 | 42 | 1300 | .039 | 88 | 12 | 0 |
| 920 | 1590 | 72 | 2900 | .010 | 65 | 27.5 | 5.5 |

(1) See Figure VII-16 for location of subareas.

(2) Conduit that collects the surface drainage and dry weather flow from the subarea.

(3) Node where surface drainage and dry weather flow from the subarea are input to the TRANSPORT network (see Figure VII-17B)

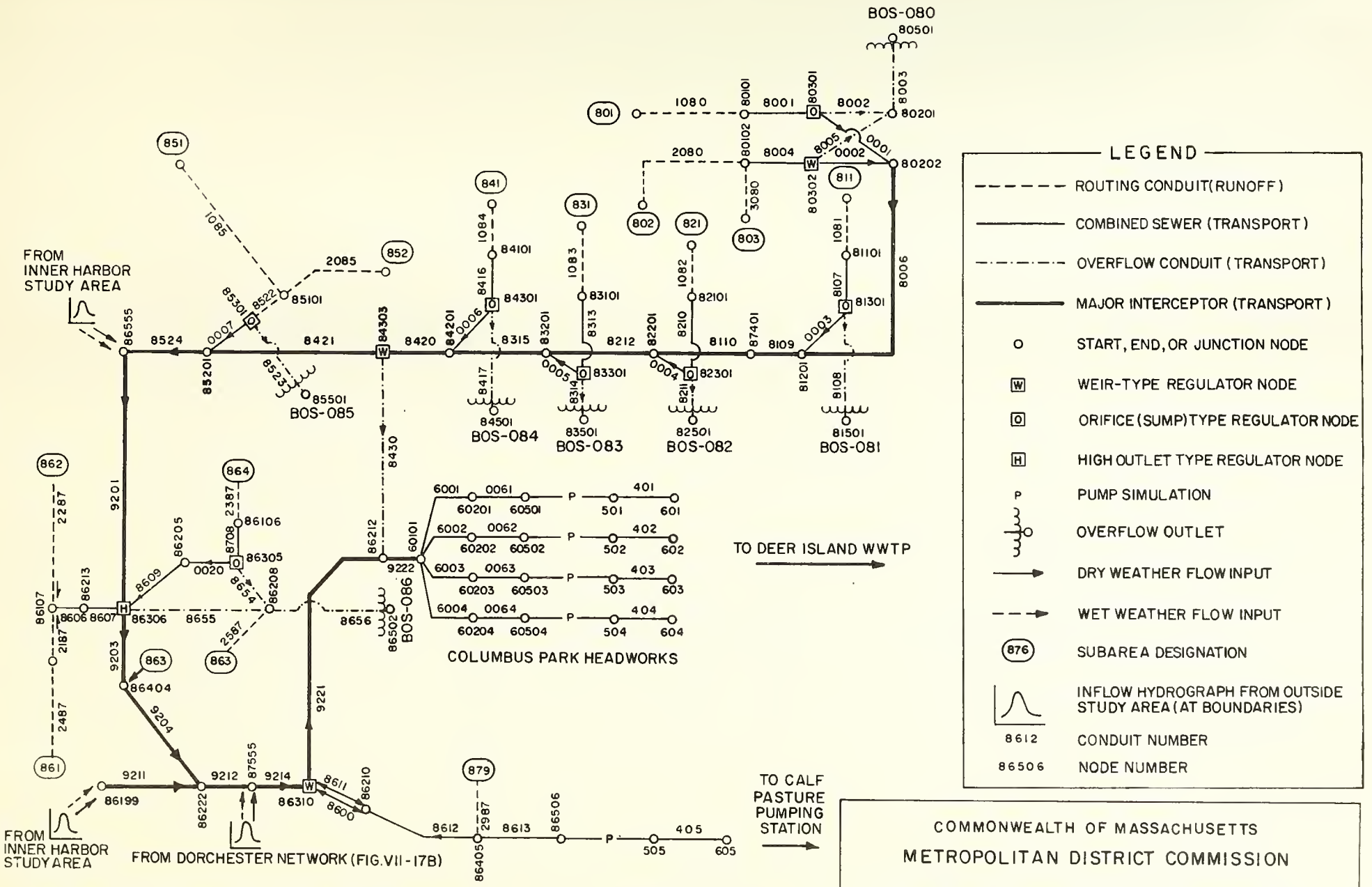


FIG. VII-17A. SCHEMATIC REPRESENTATION OF THE SOUTH BOSTON SEWERAGE SYSTEM FOR USE IN THE TRANSPORT MODEL.

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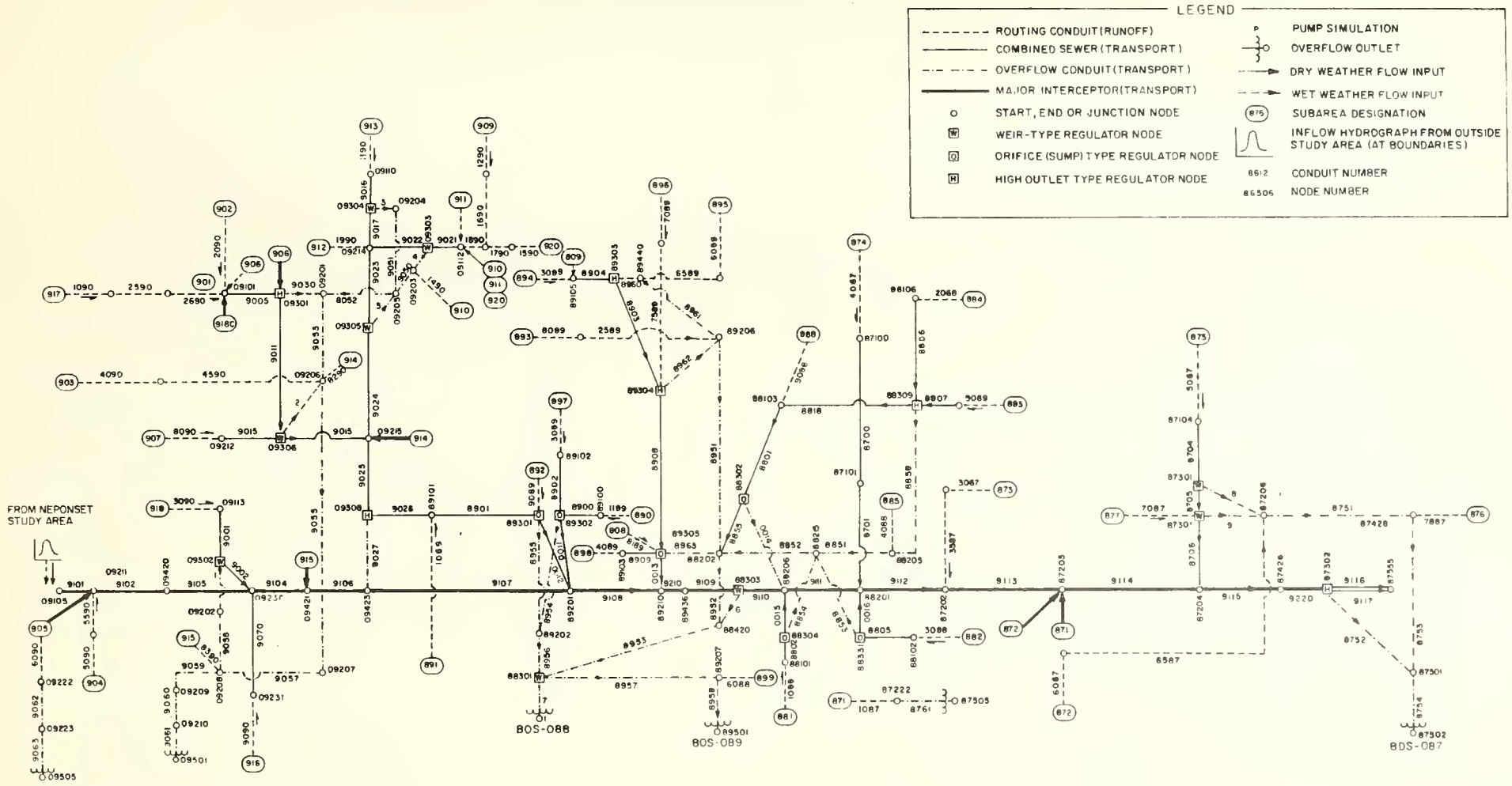


FIG. VII-178 SCHEMATIC REPRESENTATION OF THE DORCHESTER SEWERAGE SYSTEM FOR USE IN THE MODEL TRANSPORT.

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COMBINED SEWER OVERFLOW PROJECT
DORCHESTER BAY AREA FACILITIES PLAN

Table VII-7A. CONDUIT DATA FOR RUNOFF BLOCK - SOUTH BOSTON[1]

| Conduit | Diameter (ft) | Length (ft) | Slope (ft/mile) | Manning's N | Dry Weather Flow (cfs) |
|---|---|---|---|---|---|
| 1080 | 3.20 | 400 | 8.976 | .017 | .07 |
| 1081 | 1.42 | 900 | 63.94 | .017 | .56 |
| 1082 | 2.41 | 600 | 96.80 | .017 | .61 |
| 1083 | 2.02 | 600 | 66.88 | .017 | .31 |
| 1084 | 1.67 | 270 | 414.58 | .015 | .47 |
| 1085 | 5.10 | 750 | 11.088 | .015 | .24 |
| 2080 | 2.95 | 1150 | 64.44 | .017 | .13 |
| 2085 | 1.71 | 1450 | 150.75 | .017 | .31 |
| 2187 | 5.00 | 530 | 2.99 | .017 | .39 |
| 2287 | 1.71 | 570 | 23.16 | .017 | .04 |
| 2387 | 2.00 | 1070 | 37.01 | .015 | .48 |
| 2487 | 4.00 | 2430 | 27.16 | .017 | .39 |
| 2587 | 3.2 | 550 | 6.72 | .017 | 0. |
| 2987 | 10.00 | 1 | 100.0 | .013 | 0. |
| 3080 | 1.33 | 900 | 105.60 | .017 | .12 |

[1] See Figure VII-17A.

Table VII-7B. CONDUIT DATA FOR RUNOFF BLOCK - DORCHESTER[1]

| Conduit | Diameter (ft) | Length (ft) | Slope (ft/mile) | Manning's N | Dry Weather Flow (cfs) |
|---|---|---|---|---|---|
| 1087 | 2. | 200 | 176.90 | .015 | 0.0 |
| 1088 | 2. | 1600 | 12.54 | .015 | .44 |
| 1089 | 1.81 | 600 | 17.60 | .017 | .21 |
| 1090 | 1.81 | 800 | 12.67 | .017 | .14 |
| 1189 | 2.25 | 775 | 133.8 | .017 | .27 |
| 1190 | 1.5 | 780 | 139.45 | .015 | .40 |
| 1290 | 2.76 | 820 | 75.98 | .017 | .30 |
| 1490 | 4.5 | 800 | 92.40 | .017 | 0.0 |
| 1590 | 2.52 | 740 | 23.5 | .017 | 0.0 |
| 1690 | 1.8 | 1385 | 217.3 | .017 | .30 |
| 1790 | 2.23 | 1410 | 124.7 | .017 | 0.0 |
| 1890 | 2.57 | 280 | 103.71 | .017 | .30 |
| 1990 | 10. | 1 | 100. | .013 | .47 |
| 2088 | 2.25 | 350 | 42.2 | .017 | 0.0 |
| 2090 | 3.43 | 720 | 4.4 | .017 | .55 |
| 2589 | 3.64 | 1400 | 99.57 | .017 | 0.0 |
| 2590 | 2.23 | 850 | 42.24 | .017 | .14 |
| 2690 | 2.75 | 490 | 121.46 | .017 | 1.18 |
| 3087 | 1.25 | 1250 | 51.18 | .015 | .44 |
| 3088 | 1.25 | 650 | 51.99 | .017 | .21 |
| 3089 | 1.71 | 970 | 66.95 | .017 | .26 |
| 3090 | 4.03 | 1510 | 48.95 | .017 | 1.98 |
| 3587 | 1.67 | 250 | 5.28 | .017 | .53 |
| 4087 | 1.67 | 1440 | 45.47 | .015 | .32 |
| 4088 | 2.25 | 600 | 40.48 | .107 | 0.0 |
| 4089 | 1.67 | 500 | 82.72 | .015 | .43 |
| 4090 | 5.95 | 2270 | 46.75 | .017 | 0.0 |
| 4590 | 5.95 | 930 | 93.68 | .017 | 0.0 |
| 5087 | 3. | 1160 | 6.37 | .017 | .61 |
| 5088 | 1.69 | 200 | 73.9 | .017 | .26 |
| 5089 | 1.42 | 700 | 24.14 | .015 | .87 |
| 5090 | 1.0 | 775 | 68.13 | .015 | .15 |
| 5590 | 1.25 | 470 | 15.73 | .015 | .15 |
| 6087 | 1.25 | 1200 | 65.5 | .02 | 0.0 |
| 6088 | 2.5 | 600 | 76.47 | .017 | 0.0 |
| 6089 | 3.5 | 1180 | 85.02 | .017 | 0.0 |
| 6090 | 4.5 | 2280 | 10.56 | .015 | 0.0 |
| 6587 | 4. | 2100 | 10. | .02 | 0.0 |
| 6589 | 4. | 1120 | 33.94 | .017 | 0.0 |
| 7087 | 10. | 1 | 100. | .013 | .16 |
| 7089 | 2.25 | 1270 | 50.31 | .017 | .28 |
| 7589 | 2.25 | 365 | 117.17 | .017 | .28 |
| 7887 | 10. | 1 | 100. | .010 | 0.0 |
| 8089 | 3. | 900 | 44. | .015 | 0.0 |
| 8090 | 2.57 | 1070 | 67.75 | .017 | .57 |
| 8189 | 10 | 1 | 100. | .013 | .50 |
| 8290 | 10 | 1 | 100. | .010 | 0.0 |
| 8390 | 10 | 1 | 100. | .010 | 0.0 |
| 9088 | 10 | 1 | 100. | .010 | 0.0 |
| 9089 | 10 | 1 | 100. | .013 | .18 |
| 9090 | 1.25 | 675 | 10.56 | .017 | .08 |

[1] See Figure VII-17B.

Verification. TRANSPORT solves equations of gradually-varied unsteady flow based on the hydraulic properties of the given network. Thus, assuming the input hydrographs from RUNOFF are accurate and the system network approximates the prototype reasonably well, the simulation results from TRANSPORT should resemble observed field measurements for given rainfall events. However, because of the numerous uncertainties associated with the model's input data (i.e., flow at the boundaries of the Dorchester Bay study area, water surface elevation at tide gates, etc.), there is likely to be some discrepancy between simulated and observed system behavior. However, so long as observed and simulated values are reasonably well matched, one can assume that the model network and associated data are representative of the prototype system.

# CHAPTER VIII - BOSTON HARBOR WATER QUALITY MODELING

## General

The water quality modeling analysis for the Boston Harbor combined sewer overflow study investigated both the long-term (seasonal) impacts and the short-term (within storm) impacts of rainfall-related pollutant discharges to the receiving waters. Long-term evaluations have utilized a statistical analysis and modeling framework developed specifically to address questions regarding the impacts of CSO's and the long-term effectiveness of CSO abatement alternatives. A major effort of this study involved the calibration and verification of the statistical methodology in order to insure the proper modeling parameters are incorporated in the model and to show the model's ability to reproduce the observed water quality data in the Harbor. Subsequent to calibration, the statistical model was used for an analysis of the existing situation in the Harbor (Chapter IX), the screening and evaluation of CSO control alternatives (Chapters X and XI), and analysis of the final recommended plan (Chapter XII). Evaluation of the recommended plan also employed an intratidal hydrodynamic water quality model to assess the short-term impacts on water quality during design storm events. Short-term CSO impacts were evaluated using a time-variable model to simulate the individual storm events.

This chapter provides a general background for the development of both the statistical and time-variable models used for water quality simulation of the Harbor. A summary of the various calibration activities is also presented, with detailed calibration and predictive results for all runs of each model provided in Appendix D, which is included in the Inner Harbor CSO facilities plan. The Inner Harbor plan also provides a more detailed review of the mathematical framework and theory which form the basis of the Harbor models.

## Previous Work

Steady State Model of Boston Harbor. Over the years, the waters of Boston Harbor have received large quantities of domestic and industrial wastewater. Originally, all the wastewater collected was discharged untreated from the Moon Island facility. In 1952, the Nut Island wastewater treatment plant was put in operation, followed by the Deer Island wastewater treatment plant, which began functioning in the spring of 1968. At present, most of the wastewater entering the Harbor receives primary treatment and chlorination at these two facilities.

However, even with the wastewater treatment plants in operation, water quality surveys of the Harbor indicated that water use standards were being violated. Continued reduction of approved shellfish areas and restricted use of bathing areas (both due to high bacterial counts) were apparent indications of impaired water use due to wastewater entering the Harbor. To study the problem, a Federal enforcement conference was held in Boston

during May, 1968. One outgrowth of this conference was the decision to develop a mathematical model which would be used as an engineering tool to study the effects of various waste treatment alternatives on the water quality in the Harbor.

Such a model was developed, and was put forth in a 1971 report entitled Development of Water Quality Model of Boston Harbor, published by Hydroscience, Inc. (refer to Appendix D of the Inner Harbor Area Facilities Plan). The water quality variables which were considered in the analysis were the concentration of coliform organisms, biochemical oxygen demand, dissolved oxygen, digested sludge solids, nutrients, and plankton. The mechanisms included in the formulation were the dispersive effects of the tidal velocity, the advection associated with river and wastewater treatment plant discharges, the direct sources from effluents entering the Harbor, and the reaction kinetics associated with the biological activity of the substance in question. The resultant model was then verified by a comparison against observed data. When satisfactory agreement had been achieved, the model was used to evaluate the effects of various control and treatment alternatives on the water quality of Boston Harbor.

The evaluation performed in the 1971 study concluded that the coliform levels observed in Boston Harbor are due to three principal sources: the digested sludge discharged by the Deer and Nut Island wastewater treatment plants, the wastewater effluents from these facilities, and sources discharging to the Inner Harbor, including overflows from the combined sewer system. The study further concluded that the major sources of oxygen-demanding material to the Harbor were the carbonaceous and nitrogenous components of both wastewater effluents and digested sludge from the treatment facilities, the oxidation of benthic organic material, and the organic matter associated with the discharges to the Inner Harbor, including overflows from the combined sewer system.

Hydrodynamic, Time-Variable Water Quality Model. Mathematical models of tidal flow and time-variable water quality for Boston Harbor were developed by Hydroscience, Inc., and presented in a 1973 report (refer to Appendix D of the Inner Harbor Area Facilities Plan). The purpose of the work was to develop a time-variable water quality model for Boston Harbor which would properly assess the effect of noncontinuous waste discharges on Harbor water quality. Since the dominant mass transport mechanism is tidal advection, a hydrodynamic model of the pertinent portion of the Harbor was first developed.

The hydrodynamic model, referred to as WAVE, is generally applicable to shallow, non-stratified bays. Friction forces and the Coriolis acceleration were included in the model, whereas wind effects, water density gradients, and atmospheric pressure differences were neglected. No provision was made in the 1973 study for flooding or uncovering of tidal flats, and freshwater flow was considered negligible. (The model was modified for the CSO facilities planning effort so as to include the effects of wind and freshwater flow.) Physical model parameters consist of mean water depths, friction coefficients, and stage variation at the ocean boundaries. Results of the hydrodynamic model include stage and velocity at desired times within the tidal cycle, as well as a semi-graphical directional field for velocity at given times. Verification of the model was achieved in the 1973 study when agreement was obtained between velocities calculated by the model and observed velocities from NOAA current meters and USGS current

Compatible with the tidal flow model, an advective-dispersive hydrodynamic water quality model (HQ) was also developed for conservative substances and those following first order kinetics. The HQ model is coupled to the WAVE model, since tidal flows are obtained from velocity and stage data generated by WAVE. Intratidal dispersion coefficients in both longitudinal and lateral directions were assigned, with boundary concentrations and waste discharges specified as functions of time. The time-variable water quality model computes Harbor concentrations in all model segments at specified intervals over a tidal cycle.

## Steady State Model

General. The Boston Harbor study area is a tidally influenced water body combining a number of interconnected two-dimensional bay areas with one-dimensional estuaries. Four rivers supply the major portion of the advective transport through the system. These are the Charles, Mystic, Chelsea, and Neponset Rivers. The major bay areas include Dorchester Bay, Quincy Bay, Hingham Bay, and Hull Bay.

The major objective of the 1978-80 MDC CSO facilities planning study is to evaluate the impacts of combined sewer overflow on the beach and shellfish areas of the Harbor and to design the treatment facilities necessary to obtain water quality objectives. A spatial comparison between the originally developed steady-state model configuration and the major Harbor beach and shellfish areas shows the 1971 model to be inadequate for the 1978 MDC CSO study. The 1971 water quality model does not cover all areas of concern and grossly approximates major beach areas with large segment sizes. The model has therefore been expanded from 76 to 220 segments, as shown on Figure VIII-1. The major changes incorporated include expansion of the model to include portions of the Charles, Mystic, Chelsea, and Neponset Rivers, finer segment resolution at the beaches and shellfish areas, finer segment resolution near waste outfalls, and movement of the ocean boundary further away from major waste sources.

Segment volumes, interfacial cross-sectional areas and characteristic depths were calculated from National Ocean Survey maps, and are presented in Appendix D.

Advective Flows. The primary sources of non-tidal advection in Boston Harbor are the river flows emptying to the Harbor, the continuous point discharges at Deer Island and Nut Island, and the intermittent flows induced by stormwater runoff through combined sewer overflows and storm drains. All these sources of advection were incorporated into the steady-state model for each calibration period and projection, with major sources routed as shown on Figure VIII-2.

River flows were obtained from USGS records at Waltham for the Charles River, at Norwood for the Neponset River, and at Winchester for the Mystic River. Corrections to the flows to account for the ungaged drainage areas between the gaging stations and the mouths of the rivers amount to 1.3, 3.3, and 3.1 times the reported values for the Charles, Neponset, and Mystic Rivers, respectively. Flows from the Chelsea, Weymouth Fore, Weymouth Back, and Weir Rivers were estimated using an average flow per unit

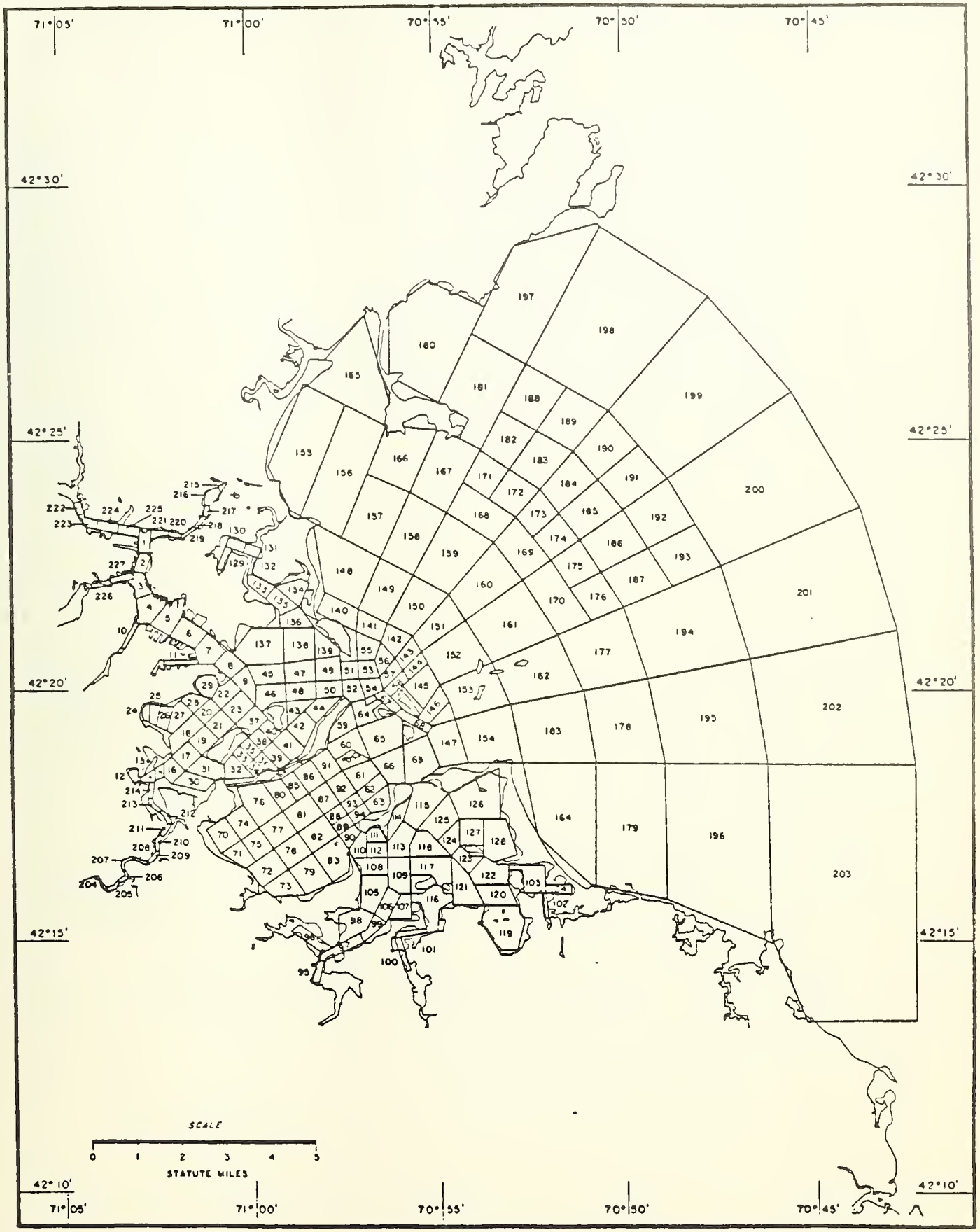


FIG. VIII – 1 STEADY-STATE MODEL SEGMENTATION

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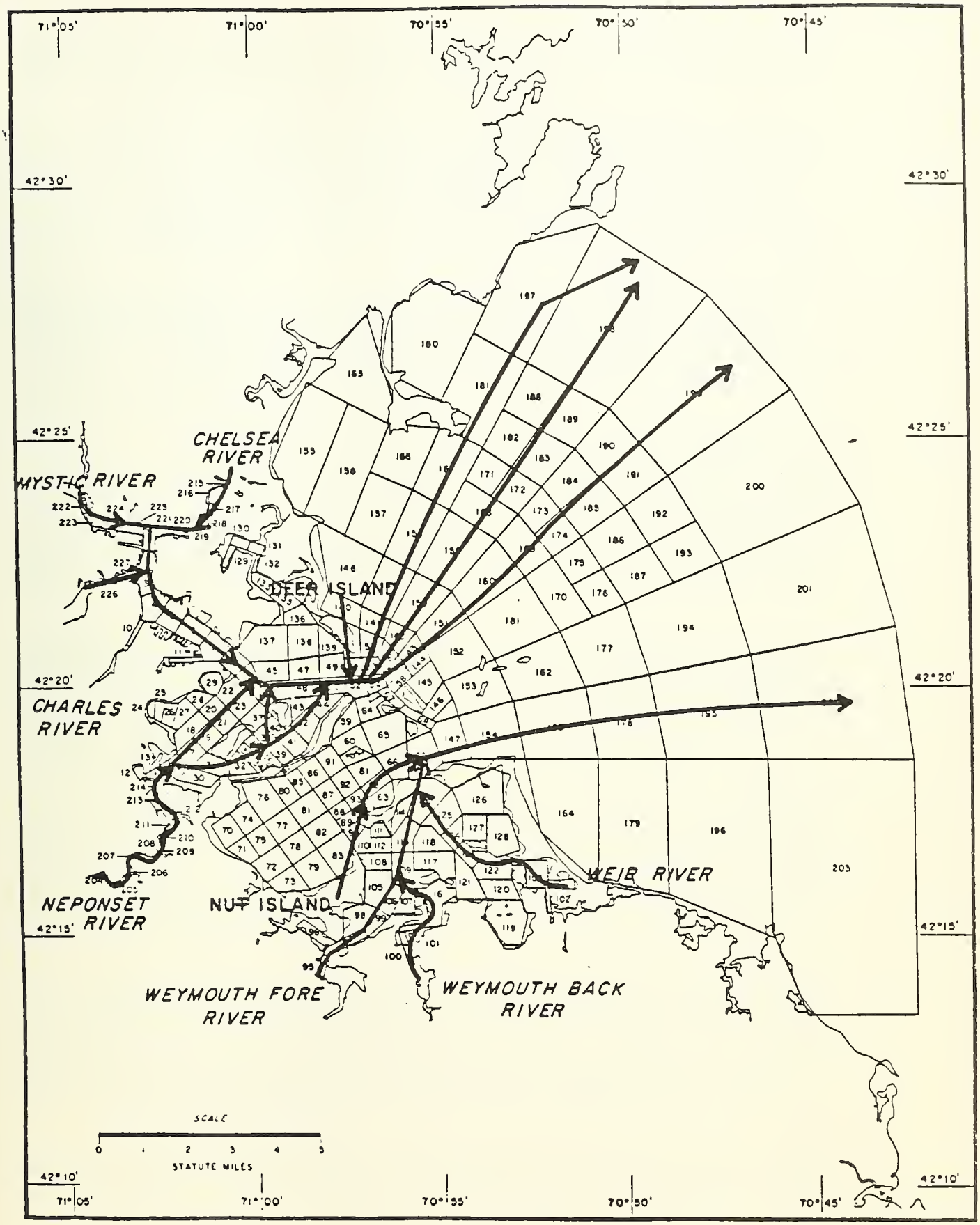


FIG. VIII-2 STEADY-STATE MODEL FLOW ROUTING

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COMBINED SEWER OVERFLOW PROJECT
DORCHESTER BAY AREA FACILITIES PLAN

drainage area from the gaged rivers. Wastewater treatment plant flows at Deer Island and Nut Island for the steady-state model calibration periods were obtained from treatment plant records.

The advective flows induced by rainfall-runoff were generated by a land-side simulator. An average summer rainfall (0.101 in/day) was used to calculate the combined sewer overflow and storm drain runoff volumes. The total rainfall-induced flow on a daily basis is 83 cfs. This flow is dispersed throughout the Harbor, and is therefore small relative to other sources of advection.

Dispersive Transport. In addition to the advective transport, the dispersive transport within the Harbor must be considered. In an estuarine system, tidal mixing due to the constant movement of the flood and ebb tides is the primary cause of dispersion. In many estuarine systems, dispersive transport due to tidal mixing exerts a greater influence on the total transport than does advection.

Salinity, as a conservative substance and an indicator of the amount of seawater in a water body, is used to measure the degree of tidal dispersion in an estuarine system. Average summer salinity concentrations for 1967 and 1968 were analyzed in the 1971 study, and supplied information on the dispersive transport for the major portion of the Harbor. Additional chloride data collected in 1978 by O'Brien & Gere Engineers supplied the basis for refinements of the dispersive transport in the Inner Harbor and Neponset Rivers.

Calibration of Transport Characteristics. The 1967 salinity data showed concentration ranges from 28.4 grams/liter at Deer Island to 27.6 grams/liter at the mouth of the Neponset River, while the 1968 data showed ranges from 31.2 grams/liter at Deer Island to 30.5 grams/liter near Thompson Island. These small salinity gradients indicate the Harbor system to be highly dispersive, with a relatively small advective transport.

The 1978 O'Brien & Gere sampling program measured chloride concentrations over the tidal cycle at each of the sampling locations for ten different survey periods. A sinusoidal regression analysis of the chloride measurements was used to calculate the average chloride concentration for a given survey at a particular location. The calculated mean concentration from the regression analysis represents the daily mean concentration of chlorides over the complete tidal cycle.

A review of the 1978 chloride data indicates a non-steady state condition for the sampling period, with the mean chloride concentration steadily increasing from August 15 through October 3, 1978. Since little or no chloride data were collected in the Inner Harbor during other calibration periods (1967 and 1968), and since nonsteady state conditions were observed during the 1978 sampling period, a time-variable model was warranted in order to analyze the dispersive transport in the Inner Harbor.

A twenty-segment time-variable model was created for this analysis. The model includes the Mystic River from the Amelia Earhart Dam, the Charles River from the Charles River Dam, the Chelsea River, and the Inner Harbor

drainage area from the gaged rivers. Wastewater treatment plant flows at Deer Island and Nut Island for the steady-state model calibration periods were obtained from treatment plant records.

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A twenty-segment time-variable model was created for this analysis. The model includes the Mystic River from the Amelia Earhart Dam, the Charles River from the Charles River Dam, the Chelsea River, and the Inner Harbor

from the confluence of the Mystic and Chelsea Rivers to the mouth. The only source of salinity considered in the analysis was the boundary at the mouth of the Inner Harbor. The boundary condition varied in time corresponding to the observed measurements of salinity collected at Pleasure Bay. Advective flows from the Mystic, Chelsea, and Charles Rivers were also varied in time according to the discharge records of the USGS. Dispersion coefficients were selected to best reproduce the chloride concentrations collected at East Boston Pier #1 during the 1978 sampling period. Through this time-variable analysis, a dispersion coefficient of 1.0 $mi^2$/day was found to be the most representative of the mixing characteristics in the Inner Harbor.

The steady-state model transport characteristics were calibrated using the receiving water chloride data collected in 1978 by O'Brien & Gere Engineers and salinity data collected in 1970 by the New England Aquarium. The average long-term concentrations were calculated for each data set. The dispersion coefficients used in the 1971 water quality model were assigned similarly throughout the major portion of the Harbor in the 1978 water quality model, with the time-variable result used in the Inner Harbor. Dispersion coefficients in areas where the model was expanded were assigned to best reproduce the available data. The advective flows used in the calibrations were average seasonal flows measured by the USGS and adjusted according to drainage area. Calculated salinity and chloride concentrations matched well with the observed 1970 and 1978 data, indicating that transport conditions within the Harbor are adequately represented by the model.

## Methodology for Analysis of Receiving Water Coliform Bacteria Concentrations

Introduction to Statistical Method of Analysis. The purpose of this analysis was to quantify the effect of intermittent runoff mass discharges on the water quality of Boston Harbor. Conceptually, the problem is straightforward: a single rainfall event produces a quantity of runoff which is discharged to the Harbor. If the quantity and quality of the discharge is known at every overflow point, the receiving water concentrations throughout the Harbor can be calculated as a function of time for an interval after which the impact is reduced to background concentrations by the natural flushing and decay mechanisms. Subsequent rainfall events are treated similarly, and the result is a time-variable simulation of the concentration at each point in the Harbor waters. There are a number of practical difficulties which limit the applicability of this method for planning purposes. The principal drawback is the excessive computation required for each simulation, since an adequate period of evaluation for a proposed plan is likely to be a number of months. A more manageable method is clearly desirable.

The rationale for adopting a statistical approach is based on the random nature of the rainfall and runoff process itself. Since this is the principal driving force for the intermittent mass discharges, it seems reasonable to analyze the receiving water response within a statistical framework. As an illustration of the method, consider the output of

results from the time-variable calculation described above. Let coliforms be the variable of concern. At a critical location such as a beach, the time history of concentration is calculated for the period of the simulation, e.g., the summer bathing season. If the daily concentrations are tabulated in time sequence, over one hundred values are available. As a summary representative of this sequence, one might calculate the various percentile concentrations, e.g., $C_{90}$, the 90th percentile concentration which is exceeded 10 percent of the time. Further, the geometric mean concentration is required since bathing standards are usually established using this statistic. For many purposes, it suffices to know just these statistics which summarize the behavior of the time-variable concentrations. This is certainly the case during the screening phase of a planning project when many types of control methods are being considered in an attempt to narrow the possible candidates for a plan which achieves water quality objectives.

The statistical method allows the calculation of these summary statistics directly without the excessive computation associated with time-variable simulations. The statistical parameters to be calculated are the mean and standard deviation of the receiving water concentrations during the period of interest. These statistics, together with a knowledge of the underlying probability density function, can be used to calculate the various percentile concentrations and the other statistical quantities of interest.

## Calibration of the Statistical Model

Data Analysis. The receiving water was monitored for bacterial concentrations at eight different locations throughout tne Harbor. The monitoring program consisted of ten surveys, with each survey collecting six to ten samples over a tidal cycle at the eight locations. These data provided the basis for calibration of the statistical model.

A sinusoidal regression analysis was used to remove the tidal effects on the measured concentrations so that a representative mean concentration could be calculated at each location for each day of sampling. The calibration of the statistical model was performed in two phases: calibration against dry weather data only in order to verify the model's ability to reproduce water quality concentration and variability caused by continuous discharges of bacteria; and calibration against all data (composite) in order to reproduce the combined effects of both dry weather discharges and rainfall-related discharges. Survey periods were considered representative of dry conditions if conducted at least 48 hours after a prior rainfall event.

Each set of data was evaluated to determine the concentration percentile of the daily log-mean concentration. The underlying probability density function of the daily log-mean concentrations for each data set were determined graphically. In all cases, the underlying density distribution was shown to be close to log-normal. At each sampling location, the concentration percentiles of the 1978 sampling period were calculated for both the dry and composite data sets.

River samples were also collected at several locations from September 12 through 16, 1978. Since these data were all collected in one week of the simulation period, they are not representative of the long-term water quality, and it was not possible to calculate long-term concentration percentiles. Consequently, each daily log-mean concentration was calculated and plotted with the subsequent calibration figures.

Dry Weather Calibration. The statistical water quality model was first calibrated using the dry weather data set in order to insure the model's ability to reproduce background water quality conditions. The major coliform sources in the Boston Harbor area during dry weather are the continuous point sources at the two major wastewater treatment plants (Deer and Nut Island) and the continuous distributed discharges of dry weather over-flow caused by faulty tide gates and regulators.

Coliform discharges from the Deer Island and Nut Island treatment plants were evaluated through a statistical analysis of a three-year record of daily treatment plant effluent data. Dry weather overflow discharge rates were estimated by the area consultants through observation and measurements, with dry weather samples collected in order to measure the total coliform concentration in the overflows.

It is difficult to directly quantify the variability of dry weather overflow loading rates due to the influence of fluctuations in water use, the rate of breakdown of regulators and tide gates, and the degree of maintenance of the sewer system. For this reason, the receiving water quality data collected during dry weather conditions were used to estimate the time and mass variability associated with the continuous sources.

A constant coliform die-off rate of 3.0/day was assigned to all areas of the Harbor, which is the same rate used for the 1967 calibration period in the 1971 study. It should be noted that the calculated variability of the receiving water should always be less than that observed, since the statistical approach does not consider certain sources of variability, including temperature fluctuations, wind effects, and changes in transport conditions. The calculated dry weather receiving water concentration percentiles were based on the mean and standard deviation computed through the statistical methodology. A log-normal density distribution, as observed in the data, was assumed for computation of the concentration percentiles.

Composite Calibration. Storm-related wasteload characterization for Boston Harbor was based upon concentration data gathered during the MDC CSO study used in conjunction with detailed land simulators. These simulators (e.g. the model SEMSTORM used by the Dorchester Bay Area consultant, described in Chapter VII) have the ability to simulate the mass of a pollutant discharged to the Harbor for each storm event during a specified time period. The models use the rainfall records and the observed concentration of the pollutant in each area to generate the loading rates.

The results of the land simulator computations performed by each study area consultant were transmitted to the HMC in the form of exceedance curves (See Figure VII-14). That is, for each outfall, the percent of time the mass discharged is exceeded during the simulation period was computed and

plotted on probability paper. Along with the exceedance curve for each outfall, the total number of overflow occurrences, as well as the total number of days in the simulation period were supplied. This information allowed the computation of the mean daily loading rate, the event-to-event mass coefficient of variation, and the average time between storms and overflows.

The statistical methodology was employed to calculate the mean and standard deviation of the receiving water coliform concentration due to the rainfall-related loads only. The receiving water statistics calculated for the 1978 simulation period are, therefore, the mean and standard deviation due to the continuous dry weather discharges, and the mean and standard deviation due to storm-related discharges. The receiving water means and standard deviations due to both the dry weather sources and the storm related sources were then combined in order to yield a total distribution. It was assumed that the underlying density distributions of the receiving water concentrations caused by both the continuous dry weather sources and the storm-related sources were log-normal. Corresponding concentration percentiles of the total distribution were determined from the statistics of the two-component distributions as well as the estimated deviations of the total distribution from log-normalcy.

The model's ability to reproduce long-term receiving water coliform distributions was further tested utilizing EPA beach data and New England Aquarium data collected during the summer of 1970. The same procedure used to reproduce the 1978 data set was employed for the 1970 information. As with the 1978 results, the calculated concentrations showed good agreement with the observed data, and appeared to substantiate the statistical methodology's ability to reproduce and predict the effects of rainfall-related discharges on water quality.

The statistical model was then employed for the screening of alternatives and analysis of the final recommended plan. Again, the rainfall-related discharges for the designated land areas of each alternative were supplied by the four area consultants to the HMC. The alternatives include: TeSe (existing treatment, existing storage), TeSmax (existing treatment, maximum storage), Inner Harbor area alternatives, Dorchester Bay area alternatives, Deep Tunnel, and Moon Island alternatives. For each alternative, the 50th, 80th, 90th, and 99th percentile total coliform concentrations in the receiving water were computed. The results of these computations at key locations in the Harbor are presented in Appendix D, and throughout this report under the discussion of each of the alternatives.

## Hydrodynamic, Time-Variable Water Quality Models

Model Geometry. The geometry and spatial extent of the hydrodynamic model (WAVE) is presented on Figure VIII-3. The seaward boundary of the model is a line connecting the ocean boundary between Deer Island and Hull. The model extends westward to include the tidal portion of the Mystic and Chelsea Rivers and the Inner Harbor to the north, and portions of the Weymouth Fore and Back River to the south. As presently constituted, there are 16 columns and 35 rows of model segments, or a total of 560 segments. Of these, 236 are water segments, with the others falling on land. All segments are square and are 2,083 feet on a side.

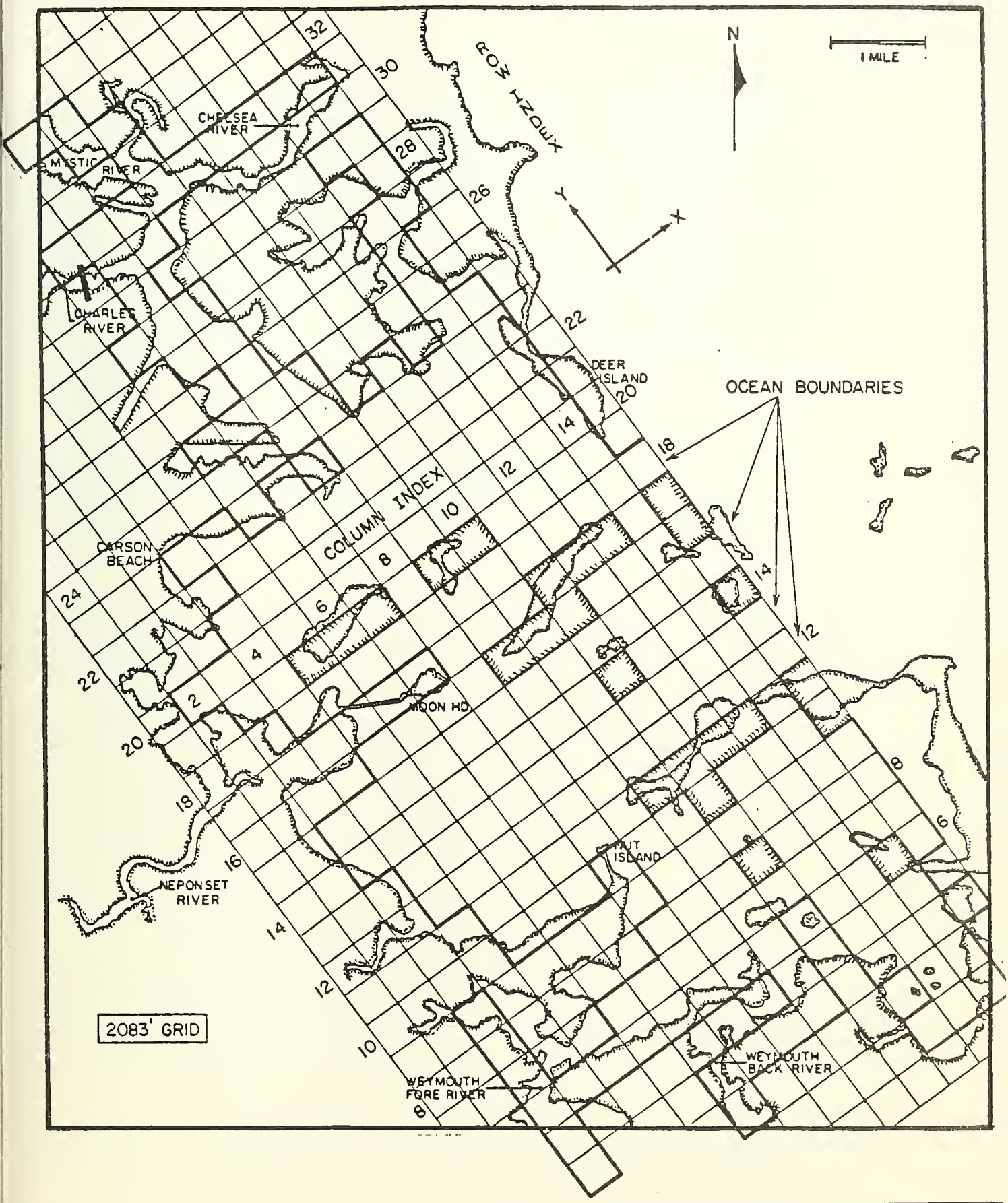


FIG. VIII-3 TIME-VARIABLE MODEL SEGMENTATION

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Segments are referred to by their row and column indices. Thus, Moon Head, a land mass, is segment (17, 7) and the water segment southeast of Deer Island is (18, 15). Although only 33 rows and 14 columns are actually used, the model must be circumscribed by a band of segments for computational reasons related to the treatment of boundary conditions. Rows one and 35 and columns one and 16 are extra boundary segments which are required when land-water interfaces bound the actual model.

The geometry of the hydrodynamic quality model (HQ) is identical with that of the hydraulic model. Input information required in addition to that needed for the WAVE model includes waste discharges and locations, decay coefficients, boundary conditions, and intratidal dispersion coefficients. During execution, program HQ assesses the stages and velocities generated by WAVE, and converts them to segment volumes and interfacial flows. Segment concentrations are then calculated by dividing the mass in a segment by its instantaneous volume.

Application to Boston Harbor. The hydrodynamic and time-variable water quality models were employed in the MDC CSO study to evaluate the short-term impacts of storm-related discharges on water quality. Specifically, the models have been used to demonstrate water quality improvements which may be expected subsequent to the implementation of the final recommended plan.

Three design storm events were selected for the analysis. The design storms, based on historical rainfall records, were the one year - six hour, three month - four hour, and the one month - four hour storm events. Each consultant submitted combined sewer overflow and stormdrain loads (flows and numbers of total and fecal coliform) resulting from each design storm to the HMC as input to the time-variable harbor model. These loads were generated by the consultants' individual detailed land-based models (the hydraulic model SWMM described in Chapter VII was used by the Dorchester Bay consultant for this purpose). For comparison purposes, the hydrodynamic, time-variable water quality model simulations included both the TeSe (existing treatment, existing storage) conditions as well as the recommended plan conditions for each storm event. The results of the simulations produced a time trace of coliform concentrations for each water quality model segment.

Prior to the design storm simulations, a sensitivity analysis was performed by the HMC to evaluate the effects of advective flow, wind, and time of the beginning of a storm event with respect to the tides on the hydrodynamics and water quality in Boston Harbor. The analysis was geared toward determining the most critical conditions for the design storm simulations. The effects of advective flow on stage height, velocity, and water quality were found to be negligible. The effects of wind on water quality were small, with a northeast wind producing a slightly greater (10 percent) impact on water quality than a southwest wind.

The analysis indicated that the critical time for the beginning of a storm event, from a water quality standpoint, is at slack tide before ebb. For purposes of completeness, advective flows as well as a northeast wind at 15 MPH were incorporated in the analysis for evaluation of the recommended plan.

The results of the time-variable simulations comparing total coliform concentrations for TeSe to the recommended plan under the three design storm conditions are presented in Appendix D, and are discussed in Chapter XII of this report along with a comparison to the long-term statistical model output.

# CHAPTER IX - ASSESSMENT OF EXISTING AND FUTURE NO-ACTION SITUATIONS

## General

In an engineering investigation such as this facilities plan, it is imperative to identify and fully evaluate the existing problem in order to accurately assess the relative benefits of each considered remedial alternative. Accordingly, since one of the primary purposes of this facilities plan is to estimate the causes of Dorchester Bay's water quality degradation and to recommend means of mitigation, the continuous long-term effects of wastewater discharges on the bay's water quality must be evaluated. The existing situation assessment is directed towards estimating the present pollutant loads entering the bay and their impact on the quality of Dorchester Bay waters based on existing system operation, including all known dry weather overflows. The future no-action situation involves the estimation of loads entering the bay, and what the subsequent impacts on the bay's water quality would be if the existing wastewater collection system were to operate at its present condition in the future. Since no major changes in land use or increases in population are expected in the study area during the planning period, the future no-action situation is, for the most part, equivalent to the existing situation relative to annual pollutant loads to the bay. Discussed in Volume II-Chapter 5 are the environmental consequences of the existing and future no-action situations.

## Quantity and Quality Of Overflow

Basis of Load Determinations. The components of CSO's consist of normal dry weather wastewater flow and stormwater runoff, combined in proportions that are dependent upon the intensity and duration of a particular rainfall event. The quantity of CSO's discharging into Dorchester Bay also depends on the available in-system storage and the system's flow conveyance capability. Both of these factors are based on the physical characteristics of the existing system, and dictate the amount of combined dry weather and stormwater runoff leaving the study area via the Columbus Park headworks, and eventually reaching Deer Island. This existing maximum conveyance and storage information, together with data on the quality and quantity of dry weather flow, the quality of stormwater runoff, and several parameters that describe drainage area characteristics, served as input to the computer model SEMSTORM to generate output information on the quality and quantity of CSO's produced within the Dorchester Bay study area. A detailed description of the model SEMSTORM, including input and output data, is presented in Chapter VII.

For the Dorchester Bay study area, the quantity and quality of CSO's, on an average annual and seasonal basis, were determined by running SEMSTORM using nearly 29 years of hourly rainfall data from the Logan Airport weather station. As described in Chapter VII, and as shown on Figure VII-11, the study area was divided into seven major sub-areas, with each sub-area further divided into areas served by combined sewers and areas served by separate storm and sanitary sewers. Only the portion of each sub-area that is served by combined sewers was simulated by SEMSTORM,

since only this area would contribute runoff to the interceptor system and cause combined sewer overflows when conveyance or interceptor capacity is exceeded. However, the total impact to study area receiving waters, as estimated by the harbor model, includes loadings from separate storm sewered areas and direct runoff areas as well as from CSO's. This is discussed in detail in Chapter VIII.

The conveyance capacity used for each of the seven SEMSTORM sub-areas includes the estimated average dry weather flow rate plus an area-weighted portion of the total excess available system capacity. This excess available capacity must be determined for each of the two hydraulically independent systems within the Dorchester Bay study area. These systems, as discussed in detail in Chapter V, are:

(1) The system tributary to the Dorchester interceptor which includes the Dorchester portion of the Dorchester Bay study area plus the entire Neponset study area, and

(2) the system tributary to tne Columbus Park headworks which includes portions of the Inner Harbor study area plus the South Boston and middle area portions of the Dorchester Bay study area.

The existing system conveyance capacity for sub-areas 5, 6 and 7 which comprise the Dorchester portion of the study area (area A3 on Figure IX-1), was based on the present downstream flowing full capacity of the Dorchester Interceptor. This existing capacity was computed assuming sediment buildup in the interceptor equal to the amount that was present in the conduit prior to the recently completed interceptor cleaning program. This capacity is approximately 91 percent of the estimated flowing full "clean" interceptor capacity of 28.2 mgd. Since sediment deposition is a continuous, on-going occurrence, the existing and future no-action conditions should reflect the effect of sediment buildup, and the resulting reduced carrying capacity. The conveyance capacity of each Dorchester sub-area was then determined by adding the estimated present average dry weather flow for the sub-area, to the area-weighted remaining capacity of the Dorchester interceptor in the following manner. The dry weather flow for each sub-area was determined using the estimated flows presented in Chapter V. The estimated average dry weather flow from the Neponset study area, which is tributary to the Dorchester interceptor and thus utilizes a portion of its carrying capacity, was added to the dry weather flows from sub-areas 5, 6 and 7, and subtracted from 25.8 mgd to determine the total remaining interceptor capacity available for conveyance of storm runoff. This total remaining capacity was then allocated to sub-areas 5 and 6 on an area-weighted basis. No excess capacity was allocated to sub-area 7 or the Neponset study area due to observed excessive sediment buildup in the southern portion of the Dorchester interceptor. Calculations and observations indicated that the available interceptor cross-sectional area through sub-area 7 is sufficient only to convey the current average dry weather flow from the Neponset study area and from sub-area 7. Table IX-1 shows the conveyance capacity estimated for each sub-area.

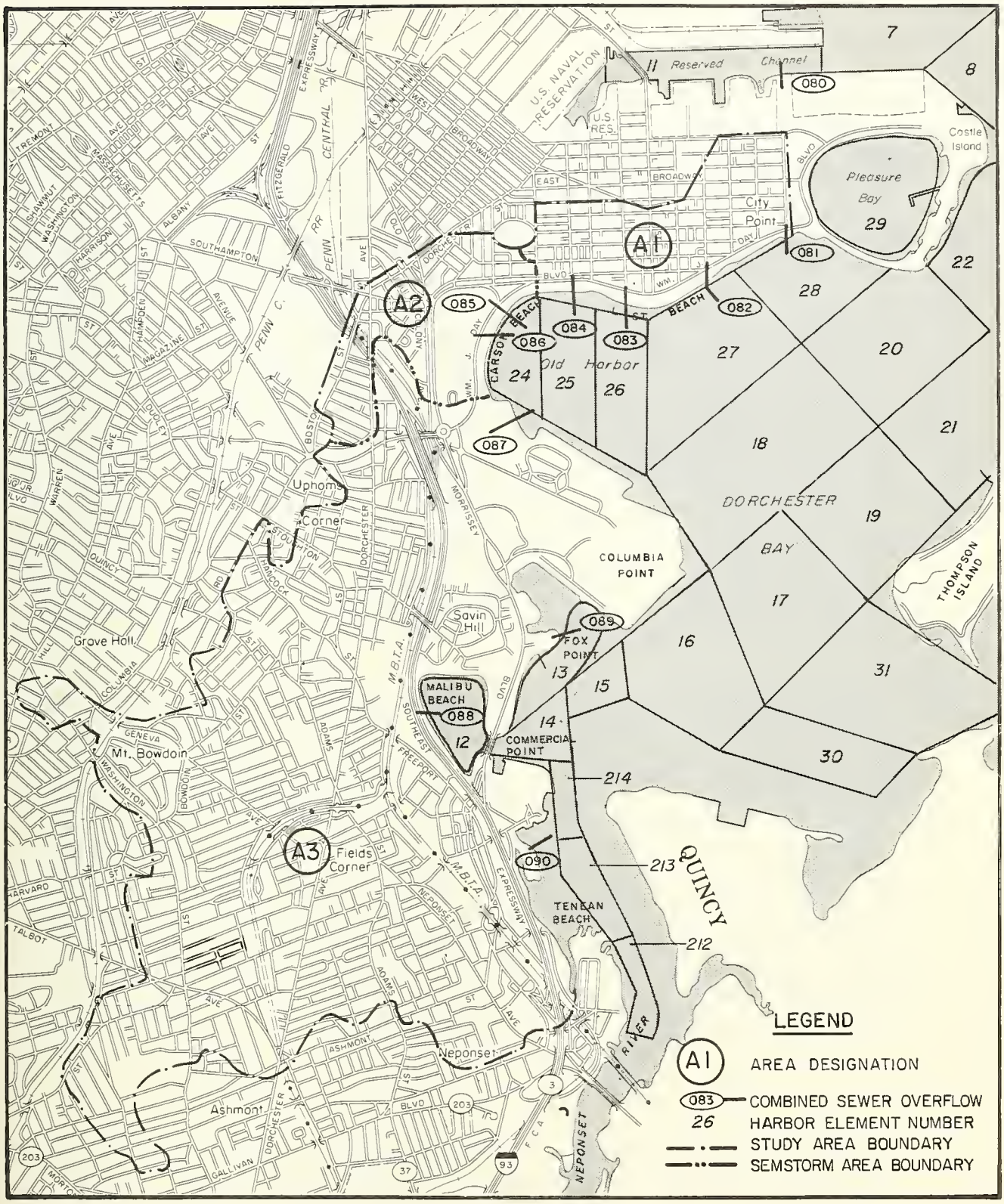


FIG. IX-1 MAP SHOWING SEMSTORM AREAS AND HARBOR ELEMENTS

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The existing system conveyance capacity for sub-areas 1, 2, 3 and 4 which comprise the South Boston and middle area portions of the Dorchester Bay study area (areas A1 and A2 on Figure IX-1), was based on area-weighted portions of the average of past flow rates (maximum) at the Columbus Park headworks. From examination of flow records at the headworks for 1977 and 1978, the average of the recorded maximum daily flows is 106 mgd. Subtracting the Dorchester area conveyance capacity of 25.8 mgd, results in 80.2 mgd of the available conveyance capacity from the remaining area tributary to the headworks. The average dry weather flow from this remaining area is estimated to be 50.7 mgd as shown on Figure V-5. (The methodology used to generate this flow is also discussed in Chapter V.) Thus, the difference between the available conveyance capacity of 80.2 mgd and the estimated average dry weather flow of 50.7 mgd, represents the remaining available system capacity of 29.5 mgd which is available for conveyance of storm runoff to the Columbus Park headworks. This was then allocated to each major tributary area to the headworks, with the exception of the Dorchester portion (including Neponset) of the Dorchester Bay study area, on an area-weighted basis. Accordingly, the existing system conveyance capacity for sub-areas 1, 2, 3 and 4 (South Boston and middle area) was estimated for each area by adding the estimated average dry weather flow, to the area-weighted portion of the remaining available system capacity (29.5 mgd). Table IX-1 indicates the results and shows the estimated existing available in-system storage volume for each sub-area.

In-system storage volume is based only on available interceptor storage within each sub-area, since storage in the system's major trunk sewers located upstream of regulators is not available above the level of the overflow weir or overflow pipe invert at a regulator. Flow exceeding this level would overflow to a CSO conduit and eventually to Dorchester Bay. Thus, it was assumed for SEMSTORM input purposes, that the only significant in-system storage is that which is available in the study area's interceptor system. Storage volume for each of the seven sub-areas was estimated by determining the difference in cross sectional area between the interceptor flowing full and the interceptor flowing at the sub-area's estimated system conveyance capacity, as determined by the procedure previously discussed. The resultant cross-sectional area was then multiplied by the length of interceptor that is within the sub-area. This volume, in cubic feet, was converted to gallons of available storage for each sub-area as shown in Table IX-1. For the reasons cited previously in this section, no excess available storage capacity was allocated to sub-area 7.

In summary, the present estimated quantity of CSO's generated from each sub-area (utilizing the model SEMSTORM) is based, in part, on the system's flow conveyance capacity and the existing available in-system storage volume as presented above. These parameters are the variables used to simulate various storage/treatment alternatives during the screening process. This screening procedure is presented in Chapter X. Several additional parameters which describe each sub-area physically, as well as hydrologically, also serve as input to the model. These parameters, which remain constant for all SEMSTORM runs, including the screening of alternatives, are utilized by SEMSTORM to estimate the

TABLE IX-1 - ESTIMATED EXISTING SYSTEM CONVEYANCE CAPACITY AND IN-SYSTEM STORAGE VOLUMES FOR EACH SEMSTORM SUB-AREA

| SEMSTORM Sub-Area[1] | Area, Acres | Estimated Ave. Dry Weather Flow, MGD[2] | Area-Weighted Portion of Excess Avail. System Capacity, MGD | Total Estimated Conveyance Capacity, MGD | Total Estimated In-system Storage, Gallons |
|---|---|---|---|---|---|
| 1 | 47 | 0.23 | 0.099 | 0.329 | 4,900 |
| 2 | 31 | 0.20 | 0.046 | 0.246 | 9,600 |
| 3 | 142 | 1.06 | 0.270 | 1.330 | 150,000 |
| 4 | 129 | 1.03 | 0.282 | 1.312 | 315,000 |
| 5 | 174 | 1.20 | 3.970 | 5.170 | 135,000 |
| 6 | 310 | 2.31 | 7.530 | 9.840 | 82,000 |
| 7 | 555 | 6.05 | 0[3] | 6.050 | 0[3] |
| Total | 1,388 | 12.08 | | | 696,500 |

Notes

[1]Refer to Figure VII-11 for a map showing the sub-areas.

[2]Refer to Chapter V. The total of 12.08 mgd for the entire study area does not include the estimated 0.59 mgd contributed from Columbia Point, since that area is served by separate storm and sanitary systems and, accordingly, was not included in SEMSTORM.

[3]Due to excessive sediment buildup in the Dorchester interceptor through sub-area 7, the available cross-sectional area is sufficient only to convey the present estimated average dry weather flow, thus no excess capacity exists for storm flow conveyance or for storage - see text.

amount of runoff that enters the sewerage system from each sub-area drainage basin, and were previously described in detail in Chapter VII. The SEMSTORM input data and the methodology that is employed to estimate CSO pollutant loads to Dorchester Bay was also discussed in Chapter VII. A summary of all quality parameters for SEMSTORM was presented in Table VII-5.

Existing Situation. Since some adjacent sub-areas have similar physical and hydrological characteristics, it was possible to combine the South Boston and Dorchester sub-areas into three major SEMSTORM subareas. As shown previously on Figure IX-1 and in Table IX-2, these sub-areas are designated as A1 for South Boston, A2 for the middle area between South Boston and Dorchester, and A3 for the Dorchester portion of the study area.

Referring to Tables IX-1 and IX-2, and Figure IX-1, area A1 is comprised of SEMSTORM sub-areas 1, 2 and 3, and has a total combined sewered area of 220 acres. It includes, for the most part, the area tributary to the south branch of the South Boston interceptor and CSO outlets BOS-080, BOS-081, BOS-082, BOS-083 and BOS-084. Area A2 represents SEMSTORM sub-area 4, and has a combined sewered area of 129 acres which is tributary to the south branch and the trunk section of the South Boston interceptor. This area is served by CSO outlets BOS-085 and BOS-086. Area A3 represents an aggregation of SEMSTORM areas 5, 6 and 7 and has a total combined sewered area of 1,039 acres. It includes the entire Dorchester portion of the study area which is tributary to the Dorchester interceptor, and is served by CSO outlets BOS-087, BOS-088, BOS-089 and BOS-090.

The total estimated existing conveyance capacity and in-system storage volume for each of the three areas as shown on Table IX-2, was determined by adding the values given on Table IX-1 that correspond to each area. For example, the conveyance capacity for area A1 consists of a summation of conveyance capacities for SEMSTORM sub-areas 1, 2 and 3 which total 1.90 mgd. In-system storage volume for each of the three areas was determined in the same manner.

A tabulation of the results generated by SEMSTORM using the 29 year rainfall record is shown on Table IX-2 for each of the three major sub-areas. These results represent average CSO frequency, volumes and pollutant loads that discharge to Dorchester Bay waters from each of the three areas. Results show that under present system operating conditions and hydraulic capabilities, CSO's occur on an average about 55 days per year, resulting in CSO volumes per overflow day ranging from 0.84 mil. gal. from area A2 to 4.16 mil. gal. from area A3. The total average volume produced per overflow day from the entire Dorchester Bay study area is 6.18 mil. gal. Pollutant loads generated from each area are given as average values per overflow day and per year for each area. Results generally reflect the differences in size among the three areas, with area A3 contributing most of the pollutant load and volume. Averaged over the entire year, the rate to the harbor from the CSO's is about 0.9 mgd.

TABLE IX-2

SEMSTORM INPUT DATA AND RESULTS OF THE EXISTING AND FUTURE NO-ACTION SITUATIONS

| | South Boston | Middle Area | Dorchester |
|---|---|---|---|
| Area designation[1] | $A_1$ | $A_2$ | $A_3$ |
| Combined sewered area (acres) | 220 | 129 | 1,039 |
| Conveyance Capacity (mgd) | 1.90 | 1.31 | 21.06 |
| In-System Storage volume (mg) | .165 | .315 | .217 |
| Overflow days per year | 62.4 | 49.8 | 53.1 |
| Ave. overflow volume per overflow day (mg) | 1.18 | 0.84 | 4.16 |
| Ave. TSS per overflow day (lbs) | 1,380 | 894 | 4,760 |
| Ave. $BOD_5$ per overflow day (lbs) | 748 | 488 | 2,590 |
| Ave. total coliform per overflow day (# x $10^{12}$) | 248 | 169 | 873 |
| Ave. overflow volume per year (mg) | 73.7 | 41.8 | 220.7 |
| Ave. TSS per year (lbs) | 86,100 | 44,500 | 253,000 |
| Ave. $BOD_5$ per year (lbs) | 46,700 | 24,300 | 137,000 |
| Ave. total coliforms per year (# x $10^{12}$) | 15,500 | 8,410 | 46,300 |

[1] See Figure IX-1

Future No-Action Situation. As previously discussed, the future no-action situation is, for the most part, equivalent to the existing situation relative to CSO quantity and quality. This situation assumes the future continuous use of the present system with its existing conveyance capacity and in-system storage volume. As discussed in Chapter III, no significant population increases or changes in land use are anticipated throughout the design period, thus the physical and hydrological parameters for each sub-area are assumed to remain basically constant. Accordingly, the results shown on Table IX-2 represent both the existing and the future no-action situations.

## Estimation of CSO Impact on Dorchester Bay

The impacts of CSO's on Dorchester Bay were estimated on both a long-term and short-term basis. The long-term impacts were estimated by the harbor modeling consultant (HMC) for the existing and future no-action situations using the output from SEMSTORM in the form of exceedence curves developed from the 29-year period of record. The short-term impacts were estimated by the Dorchester Bay area consultant based on an evaluation of near-field dispersion and dilution of the CSO wastewater to ascertain the the effect CSO's have on the study area beaches. The results of the short-term impact analysis is presented later in this section.

Since the long-term impact analysis was performed by the HMC using the harbor model, the data generated by the model SEMSTORM for each of the three major sub-areas (A1, A2 and A3) was disaggregated, on an area-weighted basis, to correspond to the harbor model elements. Referring to Figure IX-1, data was generated for nine harbor model elements located adjacent to the coastline of the Dorchester Bay study area, that serve as receiving waters for discharges eminating from the area's CSO outlets. These elements are numbered 11, 13, 24, 25, 26, 27, 28, 213 and 214. For each element, four overflow exceedance curves, one each for overflow volume, total suspended solids load, $BOD_5$ load and total coliform bacteria numbers were provided to the HMC. This data was utilized by the HMC to develop input data for the harbor model. Additional information on dry weather overflows and separate stormwater discharges to each element was also provided to the HMC in order to estimate the relative long-term assessment of the impacts of each pollutant source. The assessment was based on harbor model output results for total coliform bacteria numbers, total suspended solids concentrations and dissolved oxygen levels that are estimated to be present in the receiving waters 50, 20 and 10 percent of the time. A description of the harbor model, including the methodology and criteria used to determine receiving water impacts, is presented in Chapter VIII.

Assessment Criteria. Criteria utilized for the assessment of the existing situation differs according to assessment periods (i.e., long or short-term). For the long-term assessment period, the HMC used total coliform bacteria, dissolved oxygen, and total suspended solids to indicate harbor water quality impacts. These pollutant parameters were selected because they are considered pertinent to the project's overall objectives of improving the water quality in Boston Harbor. The total

coliforms reflect the potential for communicable disease transmission or the potential establishment of an environment that is presumed to be inimical to good public health. Dissolved oxygen concentrations indicates the extent of oxidation of organic matter taking place in the bay. Solids determinations are a measure of the degree of settleability of the waste and indicates the amount of material which can be removed by physical treatment. Also, the parameters selected are considered to be conducive to water quality modeling techniques used for the project. Current SB standards for each pollutant follows:

| Parameter | SB Standard Criteria |
|---|---|
| Total coliform bacteria | Shall not exceed a median value of 700 MPN per 100 ml and not more than 20% of the samples shall exceed 1,000 MPN per 100 ml during any monthly sampling period. |
| Dissolved oxygen | 6.0 mg/l minimum |
| Total suspended solids | Not to exceed the recommended limits on the most sensitive receiving water use. |

Under the short-term assessment conditions, the intent is to determine what the receiving water quality is during or immediately following an overflow event at the overflow outlet and the more immediately affected surrounding area. This determination was made by estimating the extent of dispersion of the overflow as a function of time. Such an evaluation is referred to as a nearfield analysis. Since in the Dorchester Bay area most overflow outlets are located in the vicinity of beaches, the most important criteria is total coliform bacteria; the criteria used by regulatory agencies for beach closings. Other criteria are also used such as aesthetics, color, and oil and grease, since Class SB waters are used for primary and secondary contact recreation.

Long-Term Assessment. The HMC quantified the effect of intermittent runoff mass discharges on the water quality of Boston Harbor. Because of the random nature of the rainfall and runoff processes, the analysis of the receiving water response was made within a statistical framework. The statistical parameters calculated are the arithmetic mean and standard deviation of the receiving water concentrations during the period of interest. The statistics, together with a knowledge of the probability density function (i.e., total coliform bacteria are assumed to have a log-normal distribution), enable the percentile concentrations during a particular season to be computed. This method is particularly useful when modeling time variable concentrations such as coliforms. Daily concentrations are tabulated in time sequence, and percentile concentrations derived: C80 - 80th percentile concentration which is exceeded 20 percent of the time. Further, the geometric mean (i.e., C50) is presented when discussing coliforms, since bathing standards are established using this statistic. These statistics summarize the behavior of the time variable concentrations, and are used to assess the existing situation on a long-term basis.

It should be noted that current SB water quality standards do not dictate a minimum number of samples to be taken during a monthly sampling period, and the results presented hereinafter are based on daily long-term statistics.

Shown in Table IX-3 are the total coliform results calculated by the harbor model for the existing situation, including dry weather overflows. The total coliform standard is compared with the calculated values for the 50th and 80th percentile concentrations in harbor model segment numbers corresponding to the five beach locations, as shown on Figure IX-1. It should be noted that concentrations shown are for the centroid of the harbor model segment, and not at the beach's swimming area. Approximated concentrations of total coliform within the swimming areas for a typical storm event are presented during the discussions of short-term assessments that follows. Referring to Table IX-3, at the 50th percentile concentrations, two locations (Malibu Beach and Tenean Beach) do not meet the standards, and at the 80th percentile concentrations, four (Carson, "L" Street, Malibu and Tenean Beaches) of the five locations have unacceptable calculated values.

Presented in Table IX-4 is a disaggregation of the harbor model's calculated total coliform results for the existing situation by pollutant source. Three possible sources of total coliform are used: dry weather overflows (DWO), combined sewer overflows (CSO), and other sources such as direct storm runoff. Calculated total coliform concentrations are shown based on which sources are removed. For example, refer to the Tenean Beach location for the 50th percentile concentration. From all sources, the calculated concentration is 5,200 per 100 ml; with only DWO removed, the calculated concentration is 400 per 100 ml; if CSO's were removed, the concentration would be 1,300 per 100 ml, and with both DWO and CSO removed, the concentration is calculated to be 40 per 100 ml. Since the 50th percentile concentration is equal to the geometric mean of the calculated values, the sum of individual source concentrations does not equal the value shown from all sources, as it would if arithmetic means were used. The most important conclusion made from the results shown in Table IX-4 is that dry weather overflows have a substantial impact on the quality of Dorchester Bay waters.

Shown in Table IX-5, is the percent of total calculated coliform concentration contributed from several pollutant sources, including adjoining project areas. Data presented concludes that the adjoining project area's total coliform loads do not have a significant impact on the calculated total coliform concentrations in the Dorchester Bay study area. Accordingly, when near-field effects are examined, the contribution from adjoining areas and the subsequent impact of boundary conditions can be neglected.

Harbor model calculated dissolved oxygen concentrations for the existing situation are shown in Table IX-6. Existing standards for SB classified water calls for a minimum dissolved oxygen value of 6.00 mg/l. The saturation concentrations shown have been calculated by the HMC based on 1967 observed data, and are dependent upon water column temperature and

TABLE IX-3 - HARBOR MODEL TOTAL COLIFORM CALCULATED RESULTS FOR EXISTING SITUATION[1]

| Location[2] | Harbor Segment No. | 50th Percentile Concentrations | | 80th Percentile Concentrations | |
|---|---|---|---|---|---|
| | | Total Coliform Standard[3] | Calculated Total Coliform | Total Coliform Standard[3] | Calculated Total Coliform |
| Pleasure Bay | 29 | 700 | 100 | 1,000 | 300 |
| Carson Beach | 24 | 700 | 600 | 1,000 | 1,700 |
| "L" Street Beach | 26 | 700 | 700 | 1,000 | 1,800 |
| Malibu Beach | 12 | 700 | 1,700 | 1,000 | 4,500 |
| Tenean Beach | 213 | 700 | 5,200 | 1,000 | 14,300 |



Notes

[1]From Tables 2a and 2b of Hydroscience's May 3, 1979 "Boston Combined Sewer Overflow Study - Harbor Model Parameters for the Analysis of Total Coliform, Dissolved Oxygen and Total Suspended Solids - Te Se, Te Smax" - includes dry weather overflows (DWO's). Number per 100 ml.

[2]Refer to Fig. IX-1 - for location.

[3]Refers to SB classification.

TABLE IX-4 - DISAGGREGATION OF HARBOR MODEL CALCULATED TOTAL COLIFORM RESULTS FOR EXISTING SITUATION[1]

| Location | Harbor Segment No. | Total Coliform Standard | | Calculated Total Coliform[4] | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | All Sources | | Only DWO Removed[2] | | Only CSO Removed[3] | | DWO and CSO Removed | |
| | | 50th% | 80th% | 50th% | 80th% | 50th% | 80th% | 50th% | 80th% | 50th% | 80th% |
| Pleasure Bay | 29 | 700 | 1,000 | 100 | 300 | <10 | <10 | 100 | 300 | <10 | <10 |
| Carson Beach | 24 | 700 | 1,000 | 600 | 1,700 | 70 | 500 | 500 | 1,200 | 30 | 200 |
| "L" Street Beach | 26 | 700 | 1,000 | 700 | 1,800 | 80 | 500 | 500 | 1,300 | 20 | 100 |
| Malibu Beach | 12 | 700 | 1,000 | 1,700 | 4,500 | 100 | 800 | 1,300 | 3,600 | 40 | 200 |
| Tenean Beach | 213 | 700 | 1,000 | 5,200 | 14,300 | 400 | 2,200 | 4,300 | 11,900 | 100 | 700 |

Notes

[1] From Tables 2a and 2b of Hydroscience's May 3, 1979 "Boston Combined Sewer Overflow Study - Harbor Model Parameters for the Analysis of Total Coliform, Dissolved Oxygen and Total Suspended Solids - Te Se, Te Smax".

[2] With existing known project area (total) dry weather overflows (DWO) eliminated, total coliform calculated at the location during wet weather period.

[3] With project area (total) CSO's eliminated, total coliform calculated at the location during wet weather period.

[4] MPN per 100 ml.

TABLE IX-5 - PERCENT CONTRIBUTION OF HARBOR MODEL CALCULATED TOTAL COLIFORM RESULTS FOR EXISTING SITUATION[1]

| Location | Harbor Segment No. | Percent of Total Calculated Coliform from Each Contribution[2] DWO | Inner Harbor CSO | Charles CSO | Dorchester CSO | Neponset CSO | Other (Storm, Etc.) |
|---|---|---|---|---|---|---|---|
| Pleasure Bay | 29 | 96 | < 4 | 0 | < 4 | 0 | < 4 |
| Carson Beach | 24 | 33 | < 1 | 0 | 52 | 0 | 14 |
| "L" Street Beach | 26 | 50 | < 1 | < 1 | < 38 | < 1 | < 13 |
| Malibu Beach | 12 | < 67 | < 1 | 0 | 22 | 1 | < 8 |
| Tenean Beach | 213 | 69 | < 1 | 0 | 16 | 3 | 14 |



Notes

[1] From Table 3b of Hydroscience's May 3, 1979 "Boston Combined Sewer Overflow Study - Harbor Model Parameters for the Analysis of Total Coliform, Dissolved Oxygen and Total Suspended Solids - Te Se, Te Smax" - For calculated mean steady state condition.

[2] <4 means less than 4, and greater than zero, and values based on geometric means. Therefore, total percentage from each contribution may be less or greater than 100.

TABLE IX-6 - HARBOR MODEL CALCULATED DISSOLVED OXYGEN CONCENTRATIONS FOR EXISTING SITUATION[1]

| Location | Harbor Segment No. | Mean Value, Dissolved Oxygen, mg/l | | |
|---|---|---|---|---|
| | | Standard | Saturation | Total Existing |
| Pleasure Bay | 29 | 6.00 | 8.23 | 7.83 |
| Carson Beach | 24 | 6.00 | 8.21 | 6.82 |
| "L" Street Beach | 26 | 6.00 | 8.21 | 6.83 |
| Malibu Beach | 12 | 6.00 | 8.14 | 6.43 |
| Tenean Beach | 213 | 6.00 | 8.10 | 6.48 |

Notes

[1] From Table 8 of Hydroscience's May 3, 1979 "Boston Combined Sewer Overflow Study - Harbor Model Parameters for the Analysis of Total Coliform, Dissolved Oxygen and Total Suspended Solids - Te Se, Te Smax."

salinity concentrations. The existing dissolved oxygen value was calculated taking into account oxidation and de-oxygenation rates, the reaeration rate per tidal velocity and water column depth, and benthal oxygen uptake. Nitrification and the net algal effect due to photosynthesis and respiration were neglected. The total existing dissolved oxygen concentrations calculated for the five locations exceed the current standards, but it should be noted that the values shown are for the mean value for the harbor model segment number shown. Accordingly, actual dissolved oxygen concentrations may be less than those shown in Table IX-6 in the immediate area of an overflow outlet following an overflow event.

The HMC also calculated the mean total suspended solids concentration at the five Dorchester Bay beach locations, as shown in Table IX-7. Concentrations increase from the northern to southern locations, with Pleasure Bay having the lowest concentration of 4.0 mg/l and Tenean Beach the highest of 7.3 mg/l. The components of the calculated total suspended solids concentrations are also presented in Table IX-7, showing the sources of suspended solids. Because of the amount of dilution available within Dorchester Bay, the total suspended solids values calculated for each harbor model element can be considered low and would not have a detrimental long term effect on the asthetics of the water. However, the suspended solids concentrations in the vicinity of the overflow and storm drain outlets during and immediately after a storm event would be higher than the values shown. Discussed in Volume II-Chapter 5 are the environmental consequences of the existing situation's long-term pollutant loads.

Short-Term Assessment. The purpose of assessing the situation on a short-term basis is to attempt to identify the impact of CSO's on the study areas beaches immediately following an overflow event. The preceeding section discussed average long-term pollutant parameters in the harbor model segments that the CSO's discharge into. This section will describe the physical and biological factors which affect the dilution and dispersion of the overflow wastewater in the vicinity of the outlets, and how the overflows affect the swimming beaches. The analysis of water quality in coastal areas is generally viewed as a multi-dimensional problem, having the following principle features:

1. Plume rise and initial dilution.

2. Two-dimensional spreading of diluted waste constituents within a mixing zone.

3. Advective transport of dilute wastewater mixture toward shorelines and other vulnerable areas.

If all CSO's were washed out to Massachusetts Bay, the area's swimming beaches would be subjected to little or no health hazards. However, if the overflow drifts towards shore, and limited flushing action occurs, the beaches may be in violation of health standards (i.e., excessive total coliforms) for a period of time. Accordingly, in order to assess what occurs during and after an overflow event, naturally occurring circulation patterns and their probable effect on dilution of the overflow should be evaluated.

TABLE IX-7 - HARBOR MODEL CALCULATED MEAN TOTAL SUSPENDED SOLIDS CONCENTRATIONS FOR EXISTING SITUATION[1]

| Location | Harbor Segment No. | Total, mg/l | Components of Calculated Total Suspended Solids, mg/l[2] | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| | | | Charles River | CSO | Storm Drains | Undesignated | Point Sources | DWO | WWTP's Sludge | Neponset River |
| Pleasure Bay | 29 | 4.0 | 0.8 | 0.1 | 0.2 | 0.1 | 1.3 | 0.4 | 1.1 | 0.03 |
| Carson Beach | 24 | 4.2 | 0.8 | 0.2 | 0.3 | 0.1 | 1.3 | 0.4 | 1.0 | 0.04 |
| "L" Street Beach | 26 | 4.1 | 0.8 | 0.2 | 0.3 | 0.1 | 1.3 | 0.4 | 1.0 | 0.04 |
| Malibu Beach | 12 | 4.9 | 0.8 | 0.2 | 1.0 | 0.1 | 1.2 | 0.4 | 1.0 | 0.2 |
| Tenean Beach | 213 | 7.3 | 0.7 | 0.4 | 3.2 | 0.1 | 1.1 | 0.4 | 0.9 | 0.5 |



Notes

[1] From Table 12 of Hydroscience's May 3, 1979 "Boston Combined Sewer Overflow Study - Harbor Model Parameters for the Analysis of Total Coliform, Dissolved Oxygen and Total Suspended Solids - Te Se, Te Smax."

[2] Total suspended solids loads from Mystic and Chelsea rivers are not included since their impact on Dorchester Bay area beaches is negligible.

Circulation is generally caused by tidal currents, wind shear and thermal stratification. Dorchester Bay displays a tidally controlled change in depth of about 9.5 feet on the average, resulting in relatively substantial currents, as shown on Figure IX-2 (also Volume II - Figure IV-11). The strongest currents tend to occur in the boating channels, and the overflow outlets are generally located to the northwest of them.

The currents are lowest near the outlets, and generally run tangentally to the swimming beaches. Therefore, the major tidal effect on the near-field is to slowly move the overflow back and forth along the beaches. Since turbulent mixing is a strong function of crossflow velocity, the tidal variation seems to have a minimal effect on initial dilution, on two dimensional spreading, and it only slightly influences advective transport.

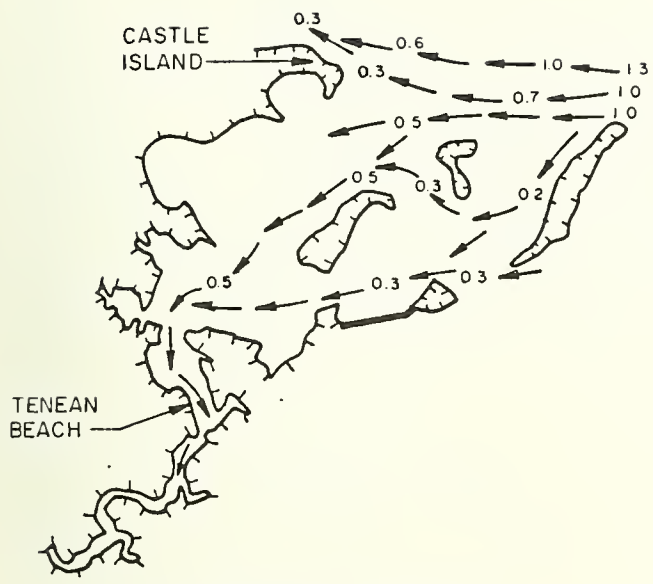
The extent of an overflow's circulation and dilution is also impacted by the wind. Shown on Figure IX-3 is the study area along with a "wind rose" that depicts the average annual and summer season's wind directions in Massachusetts Bay coastal waters. It is assumed that wind directions in Dorchester Bay are similar to those shown. Prevailing wind directions during the summer season are southwest (15 percent of time) and south-southwest (13 percent of time).

For the ten CSO outlets located in Dorchester Bay, onshore wind directions occur on an average 70 percent of the time during the summer bathing season.

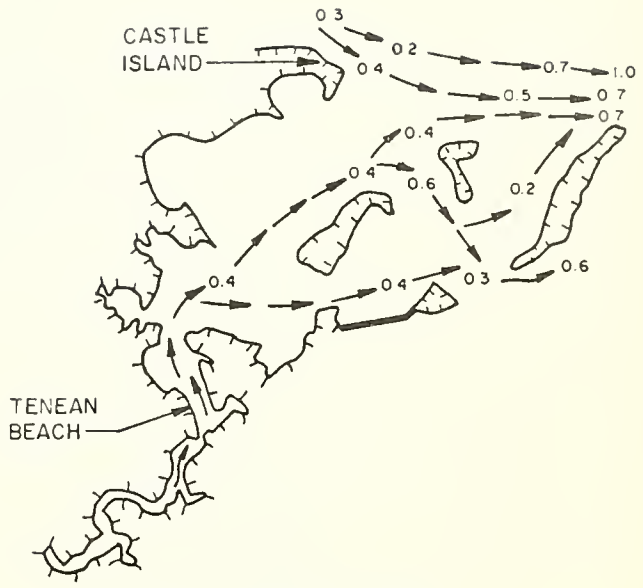
The overflow wastewater, being less dense than the bay's ambinet saline water, will rise to the surface after exiting an outlet. During onshore wind conditions, the wind shear tends to direct the overflow onto the beach. The less dense wastewater would then tend to hug the shore, allowing minimal two dimensional spreading (Figure IX-4). In the opposite case, when the wind direction is offshore, the plume continues to expand with time as it is blown away from shore. In this case, both two dimensional spreading and advective transport increase because of the increase in interface area for entrainment and the increase in ambient velocity with tidal currents. These two widely divergent scenarios indicate that both wind speed and direction strongly influence the water quality of the bathing beaches in the vicinity of overflow outlets.

Thermal stratification can effect a water body's circulation, and vertical variations in density and gradients tend to dampen out jet turbulence and inhibit initial dilution. Based on a review of historical data collected on the quality of Dorchester Bay (i.e., dissolved oxygen, salinity, temperature), and because of the bay's shallow depth, it can be assumed that complete mixing throughout the water column occurs. Thus, the effects of stratification were neglected when evaluating near field effects.

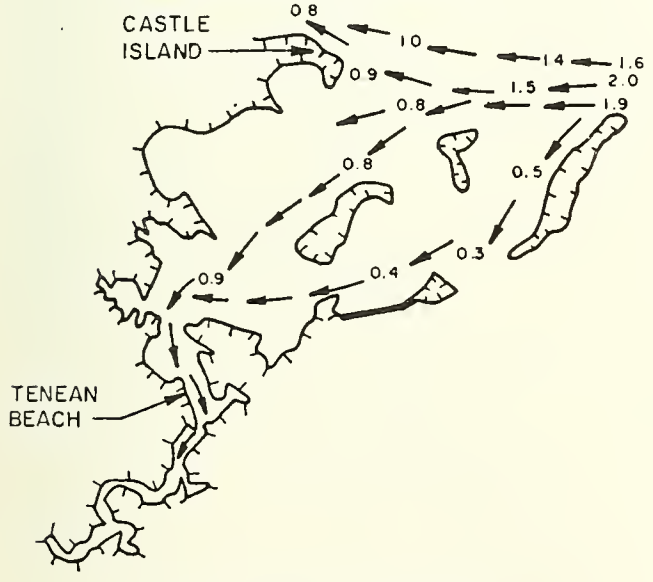
In addition to general circulation of the overflow wastewater, the effects of dilution should also be examined. Initial dilution takes place during the rise of the effluent plume toward the water surface or to some intermediate trapping level. Momentum of the discharge jet,



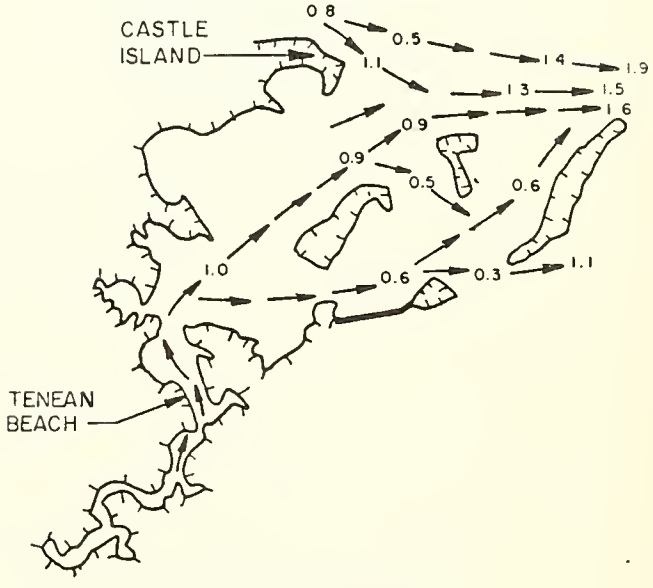

1 ½ HOURS AFTER LOW WATER




1 HOUR AFTER HIGH WATER




3 ½ HOURS AFTER LOW WATER




3 HOURS AFTER HIGH WATER

SOURCE : COASTAL GEODETIC SURVEY PUB. H504

NOTE : ARROWS REPRESENT CURRENT DIRECTION. VELOCITIES ARE EXPRESSED IN NAUTICAL MILES PER HOUR.

FIG. IX-2 TIDAL CURRENTS IN DORCHESTER BAY

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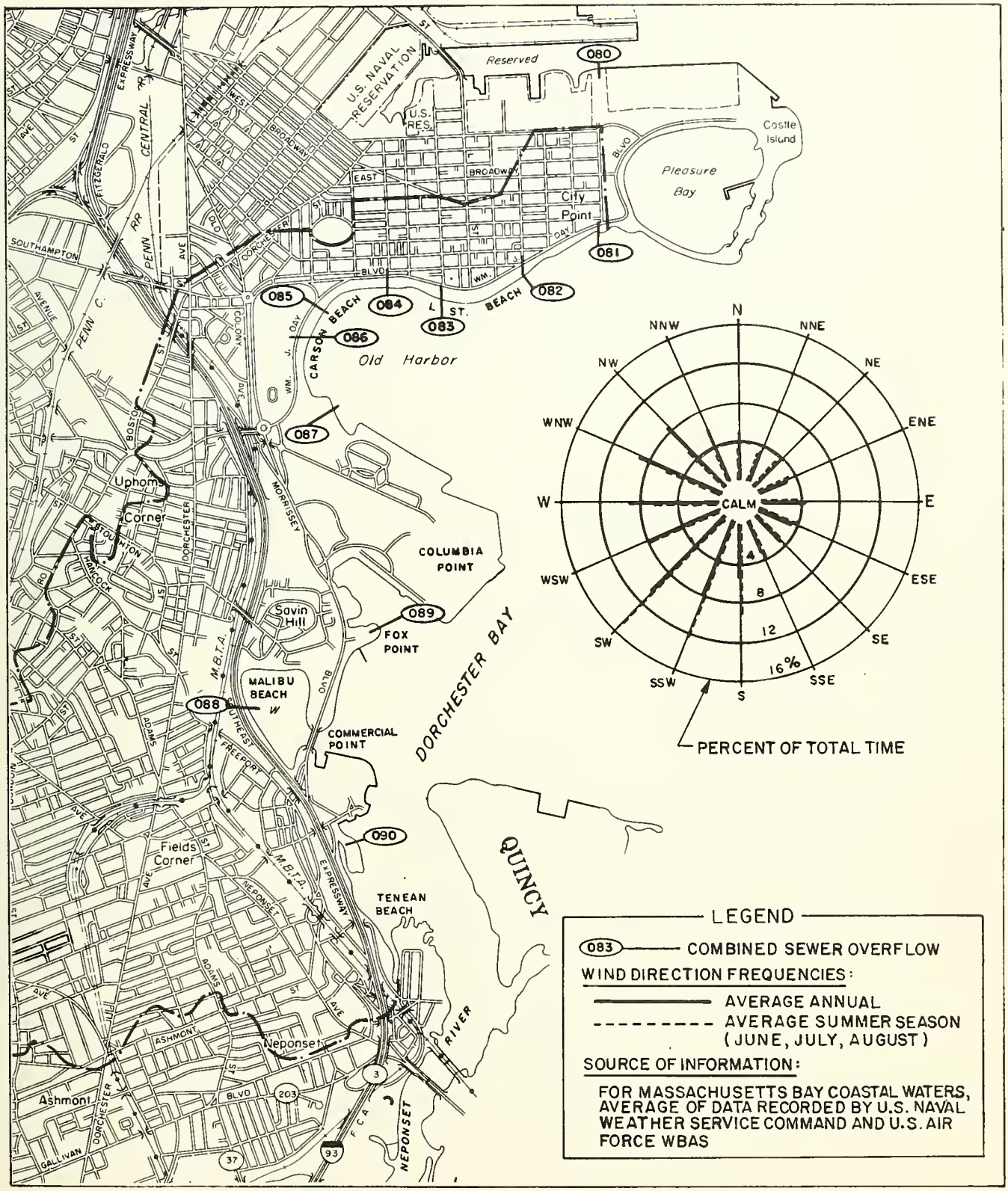


FIG. IX-3 APPROXIMATE FREQUENCY OF WIND DIRECTION IN DORCHESTER BAY

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METROPOLITAN DISTRICT COMMISSION
COMBINED SEWER OVERFLOW PROJECT
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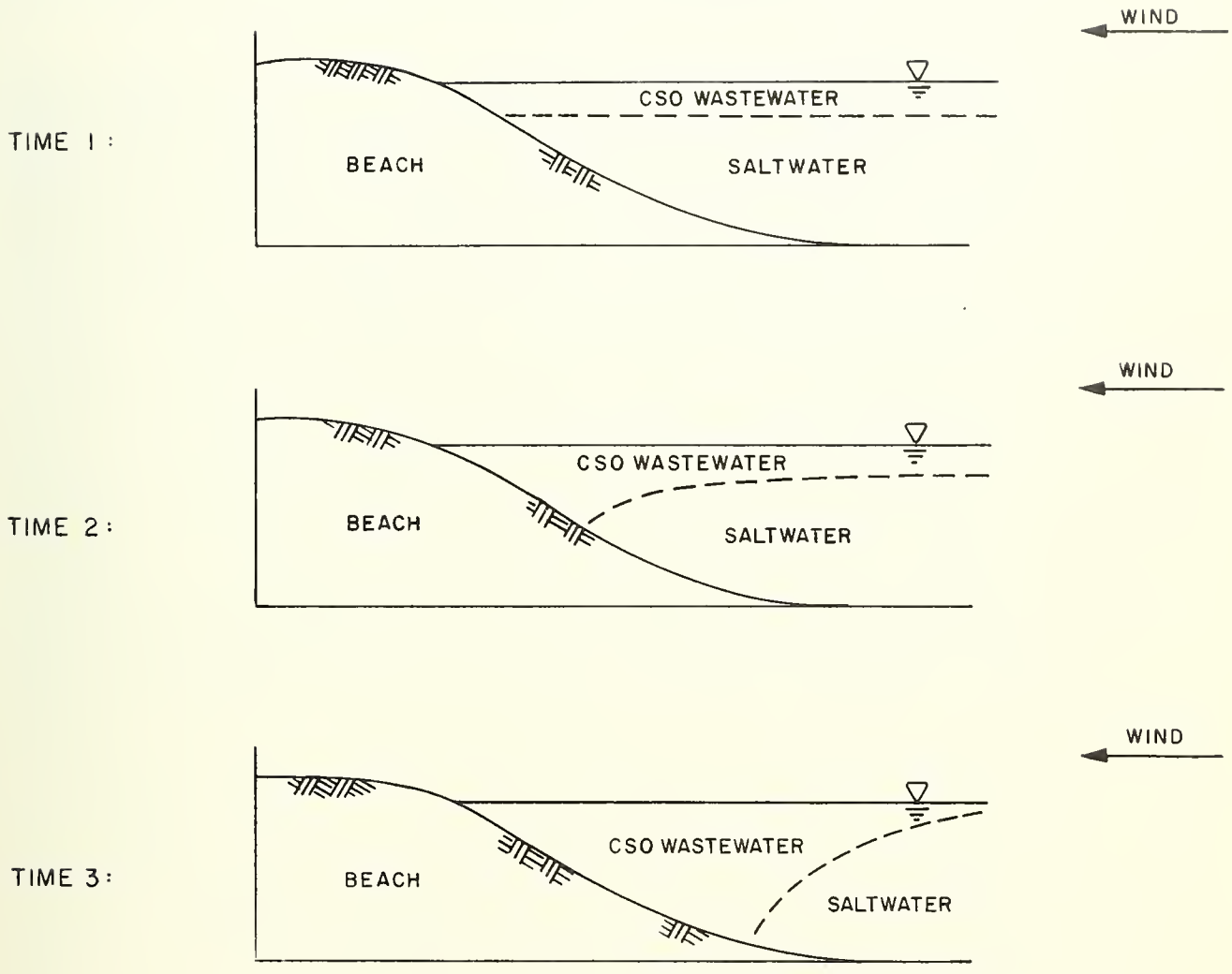


FIG. IX-4 PROBABLE RESULT OF CSO WASTEWATER PLUME DRIVEN BY ONSHORE WINDS.

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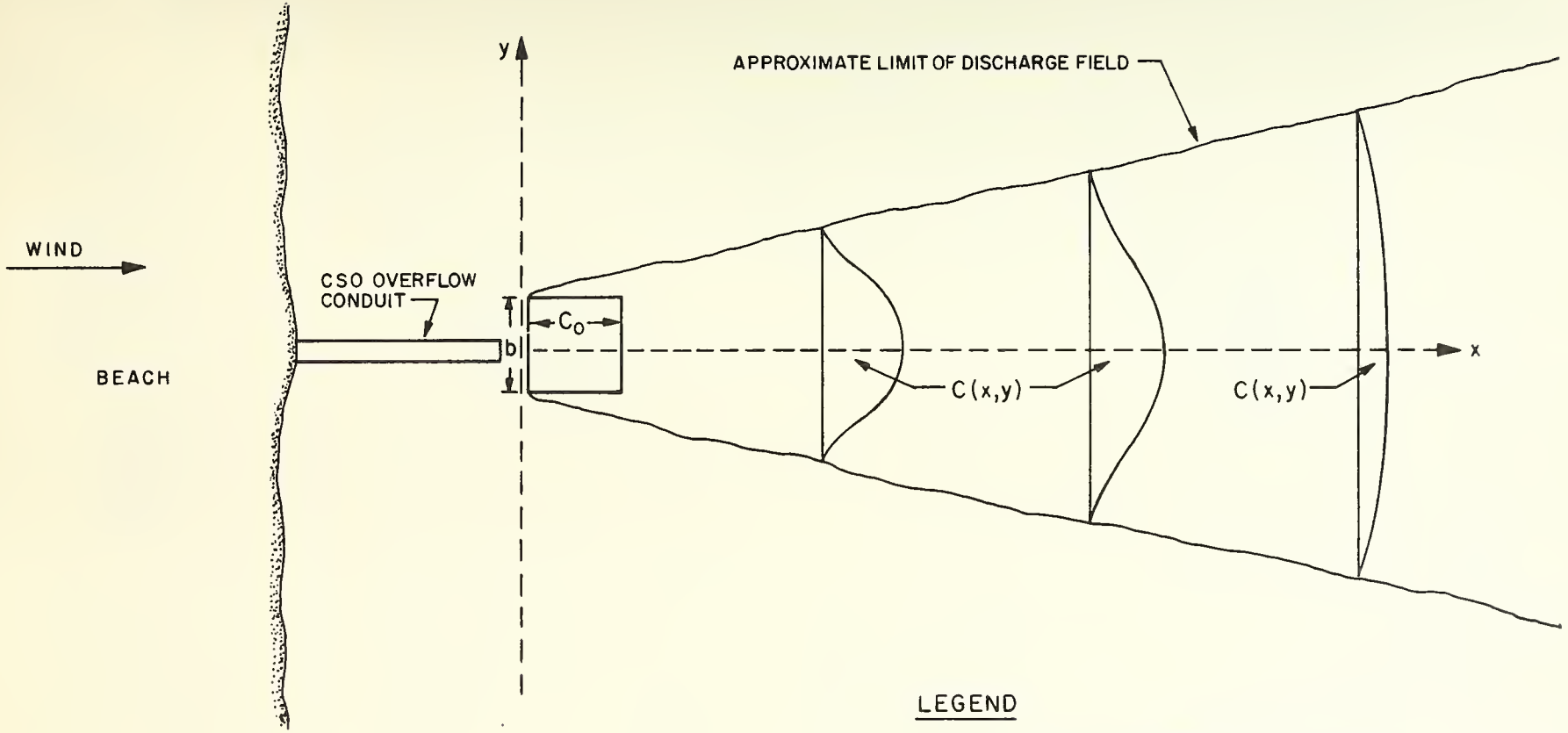
together with the buoyancy associated with density and temperature differences, produces shear stresses resulting in boundary turbulence as the plume rises toward the surface. This turbulence facilitates seawater entrainment and dilution of the effluent. The relationships between bouyancy shear and dilution are complex and dependent, to a large degree, on the characteristics of the outfall structure and the discharge itself.

To encourage maximum dilution, outfalls or outlets are usually equipped with diffusers. Basically, a diffuser forces the discharge to exit at a rate, direction and location which will maximize entrainment. The combined sewer overflow outlets in Dorchester Bay are not equipped with diffusers thus, when coupled with the shallow depth of the water column over the outlet, the resultant initial dilution is quite small. Accordingly, the concentration of the effluent when it reaches the surface is not substantially less than when it first exits the outlet pipe. In an effort to identify the range of initial dilution, Outlet No. 083 was examined for a best case situation. Assuming the outlet's average overflow (estimated to be 0.50 mg for 5 hours, or 3.71 cfs flow rate) event occurred at high tide, the initial dilution factor is estimated to be 2.0, or the effluent would loose half its strength. After the initial dilution of the effluent field is completed, dispersion (i.e., tidal currents, wind shear) spreads the field away from the outlet zone. If longitudinal and vertical mixing are assumed to be negligible, lateral dispersion is the principal factor responsible for physical dilution. This type of dilution is generally observed as shown on Figure IX-5, when the wind is offshore. The initial plume width, b, has an almost uniform concentration in the immediate vicinity of the outlet. As the field disperses along the lateral axis, a concentration gradient develops, with the concentration along the plume's centerline decreasing as it approaches the limits of the effluent field.

For considering the impact on the swimming beaches, the worst case would be a skewed onshore wind (i.e., northeast or southwest winds for "L" Street beaches) and the plume spreading with very little ambient current. For example, using the average overflow volume (750,000 gals) from any one outlet located in South Boston at the "L" Street Beach (BOS-084, BOS-083, or BOS-082) under the worst case, the approximate plume extent would be as shown on Figure IX-6. The plumes would most likely merge along the shore, resulting in one continuous body of overflow wastewater, if all outlets were to overflow simultaneously.

In addition to the physical factors affecting dilution and dispersion of the overflow effluent, biological factors should be considered. Bacteria, notably coliform, are subject to reactions which result in a concentration decay. Coliform populations undergo growth and death in a natural environment, generally assumed to follow a first-order reaction rate. The HMC adopted a die-off coefficient, K, of 3 per day for the calibration of the harbor model. The results of the field sampling program indicates that the total coliform concentration of CSO's generated from storms of the magnitude of a 1-year frequency storm event, is in the order of $2 \times 10^6$ per 100 ml. Accordingly, after one day the resulting die-off of total coiform is 100,000 MPN per 100 ml, $(2 \times 10^6) \times e^{-3}$, or 95 percent kill.




LEGEND

$C_o$ = INITIAL CONCENTRATION AT SURFACE

b = INITIAL WIDTH OF PLUME

C(x,y) = CONCENTRATION AS A FUNCTION OF DISTANCES "x" and "y"

FIG. IX-5 PHYSICAL DILUTION OF CSO WASTEWATER PLUME WITH DISTANCE FOR OFFSHORE WINDS.

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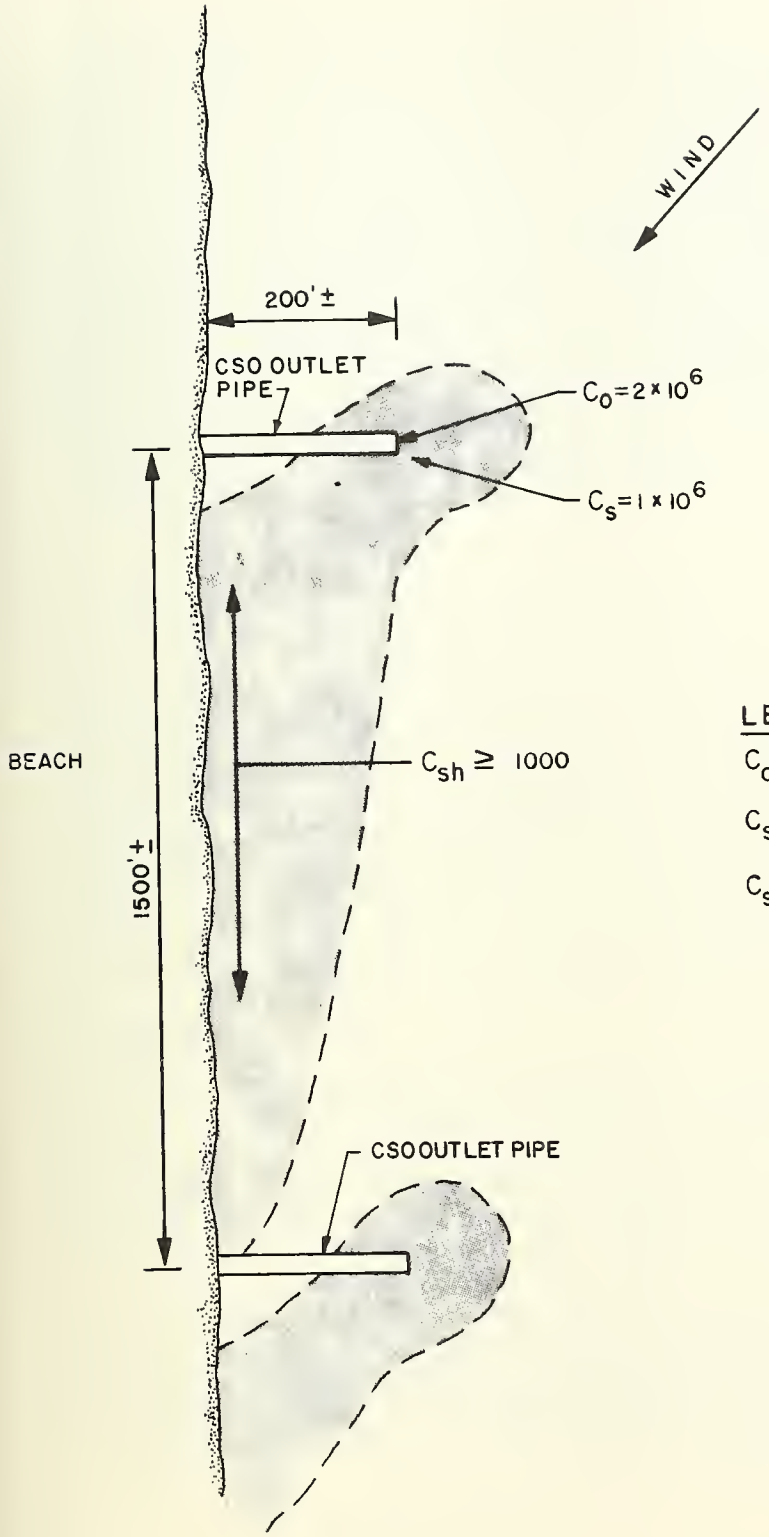


LEGEND

$C_o$ = TOTAL COLIFORM CONCENTRATION AT CSO OUTLET PIPE

$C_s$ = TOTAL COLIFORM CONCENTRATION AT AT SURFACE OVER CSO OUTLET PIPE

$C_{sh}$ = TOTAL COLIFORM CONCENTRATION AT SHORE

NOTE:

ALL CONCENTRATIONS ARE APPROXIMATE AND ARE NUMBER PER 100 ml

FIG. IX-6 APPROXIMATE DISPERSION AND DILUTION OF CSO WASTE-WATER PLUME FOR ONSHORE WIND CONDITIONS.

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Figure IX-6 shows the approximate extent of dispersion and dilution at the CSO outlets under the worst case condition (i.e. - onshore winds), taking into account biological decay factors. The effluent, as it exits the outlet pipe, has a concentration magnitude of $2 \times 10^6$ total coliforms per 100 ml. The effluent concentration is diluted to an estimated $1 \times 10^6$ per 100 ml by the time the effluent reaches the surface, which is substantially higher than allowed in SB classified waters. The total coliform concentration of the wastewater by the time it reaches the beach is dependent upon the extent of dispersion and dilution, which in turn, depends upon the wind's direction and magnitude, and tidal levels. Because of the relatively short distance between the overflow conduit outlets and the beaches, the total coliform concentration in the swimming areas during onshore wind conditions, will most likely exceed current SB standards following an overflow event. As discussed previously, onshore wind conditions occur on an average 70 percent of the time during the summer bathing season.

## Assessment of Future No-action Situation

The future no-action situation is defined as what will occur if the existing situation continues in the future, including the occurrence of continuous dry weather overflows. Since the increase in population and dry weather flows are estimated to be minimal over the planning period (Chapter III), it can be assumed that the magnitude of existing annual pollutant loads to Dorchester Bay will be the same in the future. However, as will be discussed in Chapter X, on a long-term basis, the annual pollutant loads can be reduced substantially with the elimination of dry weather overflows. The environmental consequences of the future no-action situation are presented in Volume II-Chapter V.

## Summary

Long-term annual pollutant loads entering Dorchester Bay were estimated using the model SEMSTORM. Using this information as input to the harbor model, the HMC assessed the impact of the long-term pollutant loads on the water quality of the bay. In summary, the HMC concluded that continuous dry weather overflows have the greatest effect on the degradation of Dorchester Bay's water quality.

The limits of dispersion and dilution of CSO effluent were approximated by evaluating near-field effects. It was determined that the magnitude and direction of the wind (i.e., dispersion) and the tide level (i.e. dilution) probably have the greatest effect on the water quality of the swimming beaches following an overflow event.

# X. DEVELOPMENT AND SCREENING OF ALTERNATIVES

## General

The purpose of this chapter is to discuss and present the methodology used for the development and screening of CSO abatement alternatives. During the screening phase of the evaluation of alternatives, various levels of CSO abatement measures were examined. At the completion of the screening process, a limited number of alternatives were selected to proceed into the detailed evaluation phase.

Two general types of alternatives were evaluated for the Dorchester Bay Study area:

> Structural Alternatives, which include unit processes (i.e., disinfection, storage and treatment), combined system separation and beach enclosures; and
>
> Best Management Practices (BMP's), which serve as a means of action to control urban runoff pollution at its source.

The screening process was divided into two phases. During Phase I, the purposes of current available CSO abatement alternatives were compared with the study area objectives. During Phase 2, alternatives that met the objectives were screened in greater detail and certain alternatives were selected for the detailed evaluation.

This chapter also contains the screening results of a number of joint area alternatives, some of which would utilize the existing Moon Island facilities.

Environmental issues associated with the screened alternatives are discussed in Volume II, Chapters 4 and 5.

## Study Area Alternatives

### Phase I Screening

Alternatives screened during this facilities plan are listed in Table X-1. The alternatives are based on current state-of-the-art, and were selected from the U.S. Environmental Protection Agency's (EPA's) "Urban Stormwater Management and Technology: Update and User's Guide", September 1977. As shown, the alternatives evaluated cover the full spectrum of CSO abatement techniques, and are intended to conform with the requirements of EPA's Program Requirements Memorandum (PRM) No. 75-34.

The first step of Phase I screening was to compare Table X-1 alternatives with the study area's receiving water quality objectives. This analysis provided a means of reducing the total number of available CSO abatement procedures to a limited list of issue oriented alternatives.

The overall objective of this phase of the facilities plan was to screen and select alternatives that would result in the continuance of the SB water quality classification of Dorchester Bay. Six parameters, as listed below, were selected as CSO pollutant indicators used for the purpose of defining CSO discharge limitations, and thus, the applicability of the various control techniques listed in Table X-1 to the study area.

TABLE X-1 STUDY AREA CSO CONTROL TECHNIQUES SCREENED

STRUCTURAL ALTERNATIVES

- Unit Processes

  I. Storage
     A. In-line Storage
     B. Off-line Storage
  II. Treatment
     A. Treatment at Deer Island Wastewater Treatment Plant
     B. Local Treatment of Discharges Utilizing the Following Processes:
        - Sedimentation
        - Swirl Concentrators
        - Helical Bend Concentrators
        - Screening
        - Dissolved Air Floatation
        - High Rate Filtration
  III. Disinfection

- Combined System Separation

  I. Partial Separation
  II. Total Study Area Separation

- Beach Enclosures

BEST MANAGEMENT PRACTICES

- Planning

  I. Land Use Planning
  II. Use of Natural Storage and Drainage Features
  III. Erosion Control
  IV. Porous Pavement

- Operation and Maintenance Practices

  I. Neighborhood Areas
     - Litter Control
     - Chemical Use Control
     - Street Sweeping
     - Street Maintenance
     - Management of Highway De-icing
  II. System Maintenance
     - Regulator Maintenance
     - Catchbasin Cleaning and Maintenance
     - Sewer Cleaning
     - Sewer Flushing
     - Tidegates
  III. Infiltration Reduction
  IV. Existing System Real-time Operation

- Legislation

- Total Coliform
- Total Suspended Solids
- Dissolved Oxygen
- Floatables
- Heavy Metals
- Nutrients
- Oil and Grease

The parameters selected were based on the conclusions reached during the assessment of the existing situation (Volume I - Chapter IX, and Volume II - Chapters IV and V). The impacts of the pollutant loads associated with the above indicators on the water quality of Dorchester Bay were determined. The bay's assimulation capacity for each pollutant was evaluated, along with the impact of each pollutant on the bay's current uses. The impacts of total coliform, total suspended solids and dissolved oxygen were presented in Chapter IX, as determined by the Harbor Modeling Consultant (HMC). The remaining parameters were evaluated independently by CDM.

A discussion of how each of the six pollutant parameters impacts on the study area's issues and objectives follows.

Total coliform, because of the recreational and potential shellfish harvesting uses of Dorchester Bay, is considered a pollutant that must be reduced in order to conform with the bay's SB classification. Referring to Table IX-4, if dry weather overflows are eliminated, the calculated total coliform in all areas, except for Tenean Beach, will meet SB standards on a long term basis. For the short term case, the total coliform count in the recreational and shellfish areas exceed the standards during and immediately following an overflow event. Accordingly, CSO abatement measures that result in a reduction in the number of total coliforms entering the bay during an overflow event were included during the screening process.

Total suspended solids were determined not to be detrimental to the water quality of Dorchester Bay, based on the results of HMC's long term evaluation of the existing situation. Referring to Table IX-7, only 3 to 5 percent of the total calculated total suspended solids (TSS) in the bay's water column are attributed to study area CSO's.

The southern section of Dorchester Bay has the highest concentration of TSS in the water column, and it also has the lowest depths. The volatile solids fraction, or the percent organic content, varies from 10-15 percent in the sediment at the end of the CSO outfalls in the bay, based on sampling performed in July, 1979. Thus, the majority of sediment solids appear to be inorganic, probably attributed to shoreline erosion and contributions from the Neponset River.

Because of the inorganic natue and relatively limited amounts of TSS entering Dorchester Bay from CSO's, no specific treatment processes (i.e., structural alternatives) for the reduction of TSS were considered during Phase 2 screening. However, there are inherent TSS removals associated with other CSO abatement facilities screened, thus an ultimate TSS reduction in the bay will be realized.

Dissolved oxygen concentrations calculated by the HMC were found to exceed the 6.0 mg/l minimum SB standard (see Table IX-6). Thus, structural alternatives (unit treatment processes) specifically for the purpose of reducing $BOD_5$ were not considered during Phase 2 screening. There are, however, inherent $BOD_5$ removal rates associated with other CSO abatement facilities similar to the TSS case. Short term dissolved oxygen deficiencies most likely occur in the immediate vicinity of the outlets after an overflow event, but because of tidal exchange and the assimilation capacity of the bay, the impact on the bay will be minimum.

Floatables are considered an issue in the study area because of the recreational uses of Dorchester Bay. Also, to conform with current SB standards, floatables must be removed. Accordingly, screening facilities for the removal of floatables are included as part of all CSO abatement alternatives evaluated.

Heavy metals were not modeled by the HMC. It has been assumed that the concentrations and annual volumes of heavy metals entering Dorchester Bay from the study area CSO's are approximately proportional to TSS. Surface water samples taken as part of this facilities plan (see Volume II-Table IV-3) show typical values for the six major heavy metals to range from 0.01 to 0.05 mg/l. Sediment analysis performed in 1972 (Volume II -Table IV-3) show higher values present than in the water column. The highest values were found in the yacht club areas and in the immediate vicinity of the CSO outlets. A large percentage of the heavy metals found in the sediment are most likely attributed to continuous dry weather overflows. This is probably the case at the BOS-090 outlet, where substantial depths of oil and metal base sediments are found, attributed to illegal dumping or cross connections into the CSO conveyance conduit.

Because of the toxicity associated with heavy metals, the amounts entering Dorchester Bay should probably be limited. Since heavy metals concentrations in the bay are not currently at a critical level and thus are not currently considered a study area issue, structural alternatives for the removal of them from CSO's are not warranted. Further, heavy metals are best controlled at their source. The implementation of best management practices in conjunction with other CSO abatement facilities screened, will result in a reduction in the amounts of heavy metals entering Dorchester Bay.

Nutrients are commonly contained in the storm water run-off from high density urban areas and in dry weather sanitary flows. Thus, water bodies located adjacent to urban areas are often somewhat naturally eutrophic. Existing available data does not indicate that Dorchester Bay is a eutrophic water body. However, all available means of limiting the amounts of nutrients entering the bay should be employed, in order to maintain a non-eutrophic environment. Hence, structural alternatives (unit processes) for the removal of nutrients (i.e., phosphorus and nitrogen) were not considered during Phase 2 screening. However, the elimination of dry weather overflows and the implementation of best management practices, in conjunction with the other CSO abatement facilities screened, should result in some nominal reduction in the total annual nutrient load entering the bay.

Oil and grease levels emanating from the two Dorchester area CSO's (BOS-088/089 and BOS-090) were measured during the field sampling program with the results contained in Chapter VI. In addition, during the field inspection program, two present sources of oil discharges were identified as discussed in Chapters IV, and V and shown on Table V-5. These sources, as well as any other that has not yet been identified, would be mitigated during the recommended program for DWO removal, as discussed in Chapter XII. Accordingly, it is expected that the control of oil and grease at its source would be accomplished by the DWO mitigation program as well as the implementation of best management practices which will be discussed later in this chapter. Thus, no specific treatment processes for the removal of oil and grease were considered during Phase 2 screening.

In summary, the primary CSO associated pollutants that are considered to be detrimental to the water quality of Dorchester Bay, and warrant abatement by the installation of CSO control facilities, are total coliform and floatables.

As discussed previously, on a long-term basis, it was determined by HMC that except for Tenean Beach, total coliform concentrations within the bay meet the current SB standards if DWO's are removed. However, if the Pine Neck Creek storm drain, whose tributary area consists of a 118 acre separate storm drainage system, is diverted away from Tenean Beach, then the long term total coliform concentrations would most likely meet or approach the standards even at Tenean Beach. Accordingly, if dry weather overflows are eliminated and if the Pine Neck Creek storm drain is diverted away from Tenean Beach, the overall water quality of Dorchester Bay will most likely conform with its current SB classification, except for the intermittent discharge of floatables. But, because Dorchester Bay is used predominantly for primary and secondary water contact recreational uses, during the bathing season the total coliform concentration of the water column in the primary contact recreational areas should be less than the 1000 MPN per 100 ml at all times. As discussed in Chapter IX, during and immediately following an overflow event, the total coliform concentration in the water column at the bathing areas exceeds 1,000 MPN per 100 ml.

Based on the above discussion, the main objectives of CSO abatement facilities and techniques should be to:

1. Reduce the number of overflow events during the bathing season in order to increase the use of the swimming areas;

2. Reduce the concentration of total coliforms entering Dorchester Bay in the vicinity of the bathing areas during wet weather periods;

3. Remove floatables from the CSO's, and;

4. Minimize the amounts of heavy metals and nutrients entering Dorchester Bay from CSO's and storm water runoff.

Shown in Table X-2 is a comparison of the possible CSO abatement alternatives listed in Table X-1 with the four objectives listed above. Also noted are the alternatives that were not, and the reasons for not being selected to proceed to Phase 2 of the screening process, including the reasons for the decision. In general, structural alternatives consisting of local treatment of CSO's for primarily solids removal, and BMP planning techniques and neighborhood area maintenance practices that are not applicable to the Dorchester Bay study area were not selected to be evaluated during Phase 2 screening.

Local treatment of CSO's utilizing the processes or techniques of sedimentation, swirl concentrators, helical bend concentrators, dissolved air flotation, and high rate filtration were not selected because they primarily are directed to result in the removal of suspended solids.

Proper land use planning can prevent the problems resulting from shortsighted development of open areas. Relating to stormwater management, planning can be used as a means of controlling the volume, rate, and pollutional characteristics of stormwater runoff. The concept of preventing and reducing the source of stormwater pollution best applies to developing urban areas, where encroachment is controllable, and drainage essentially conforms to natural patterns and levels.

The Dorchester Bay study area is an urbanized, fully developed area. Thus, land use planning, use of natural features, erosion control, and porous pavement are all best management planning practices that are not immediately applicable to the study area. However, these techniques should be considered when redeveloping large parcels of land.

## Phase 2 Screening

### Criteria for Phase 2 Screening

For the purpose of Phase 2 screening, the following criteria and basic assumptions were used:

1. Elimination of Dry Weather Overflows - As discussed previously, the elimination or mitigation of dry weather overflows is imperative if Dorchester Bay is to meet the SB classification. Chapter IV identified field confirmed sources of dry weather overflows, most of which resulted from lack of maintenance of the existing sewerage system. Presented in Chapter XII is a recommended program for the mitigation of dry weather overflows. For the purpose of screening, it has been assumed that this program will be implemented and the overflows eliminated.

2. Pine Neck Creek Storm Drain Relocation - Even with dry weather overflows eliminated, the long term water quality of Tenean Beach will not approach the SB classification unless the Pine Neck Creek storm drain is diverted away from the beach. Currently, during low tide periods, Pine Neck Creek is the only source of water feeding

TABLE X-2. COMPARISON OF POSSIBLE CSO ABATEMENT ALTERNATIVES WITH OBJECTIVES

| Alternatives | CSO Abatement Objectives Met with Alternative[1]: Reduce No. of Overflow Days/Yr | Reduce Total Coliform Concentrations | Remove Floatables | Reduce Heavy Metals and Nutrients | Alternative Evaluated During Phase 2 Screening Process | Reasons for Not Being Evaluated During Phase 2 Screening Process: Does Not Effectively Meet Objectives | Does Not Apply to Study Area |
|---|---|---|---|---|---|---|---|
| **Structural Alternatives** | | | | | | | |
| . Unit Processes | | | | | | | |
| I. Storage | | | | | | | |
| A. In-line Storage | F | F | F | F | Yes | | |
| B. Off-line Storage | F | F | F | F | Yes | | |
| II. Treatment | | | | | | | |
| A. Treatment at Deer Island | F | F | F | F | Yes | | |
| B. Local Treatment of CSO's | | | | | | | |
| Sedimentation | | P | P | P | No | X | |
| Swirl Concentrators | | P | P | P | No | X | |
| Helical Bend Concentrators | | P | P | P | No | X | |
| Screening | | P | F | P | Yes | | |
| Dissolved Air Floatation | | P | F | P | No | X | |
| High Rate Filtration | | P | F | P | No | X | |
| III. Disinfection | | F | | | Yes[2] | | |
| . Combined System Separation | P | P | | P | Yes | | |
| . Beach Enclosures | | P[2] | P[2] | | Yes | | |
| **Best Management Practices** | | | | | | | |
| . Planning | | | | | | | |
| I. Land Use Planning | | | | | No | X | X |
| II. Use of Natural Features | P | P | | P | No | X | X |
| III. Erosion Control | | | | | No | X | X |
| IV. Porous Pavement | P | P | | P | No | | X |
| . Operation and Maintenance Practices | | | | | | | |
| I. Neighborhood Areas | | | | | | | |
| Litter Control | | | P | | Yes | | |
| Chemical Use Control | | | | | No | X | X |
| Street Sweeping | | P | P | P | Yes | | |
| Street Maintenance | | | | | No | X | X |
| Management of Highway De-icing | | | | | No | X | X |
| II. System Maintenance | | | | | | | |
| Regulator Maintenance | P | P | P | P | Yes | | |
| Catchbasin Cleaning and Maint. | | P | P | P | Yes | | |
| Sewer Cleaning | P | P | | P | Yes | | |
| Sewer Flushing | P | P | P | P | Yes | | |
| Tidegate Repair and Maintenance | P | P | | | Yes | | |
| III. Infiltration Reduction | P | | P | | No | | X[3] |
| IV. Existing System Real-Time Operation | P | P | P | P | Yes | | |
| . Legislation | | P | P | P | Yes | | |

Notes:

(1) F = Fully meets objectives: P = Partially meets Objectives
(2) See text for explanation
(3) From Vol. 1 - Chapter 4, existing infiltration found non-excessive

the beach. Thus, during wet weather periods coinciding with low tides, the water of Tenean Beach is the storm water runoff from the storm drain's 118 acre tributary area. Accordingly, during the screening process it has been assumed that the Pine Neck Creek storm drain will be relocated to the Neponset River.

3. The Environmental Protection Agency's (EPA) PRM No. 75-34 stipulates that CSO abatement alternatives must be evaluated on the basis of receiving water impacts. As discussed previously, with the elimination of dry weather overflows and the relocation of the Pine Neck Creek storm drain, the long term water quality of Dorchester Bay would most likely meet current SB standards. Thus, the impacts of CSO's are short term, that result in the water quality of the beach areas violating the standards during and immediately following an overflow event. For the purpose of screening structural alternatives, the number of violation periods per bathing season was compared to the estimated costs required to implement the CSO abatement alternatives. During Phase 2 screening, it has been assumed that the number of violation periods approximates the short-term receiving water impacts in terms of study area issues. As part of the subsequent detailed evaluation phase, water quality impacts in terms of percentage of time exceeded will be presented.

## Structural Alternatives

### Unit Processes

General-Storage and Treatment. Control of pollution contributed from CSO's can be accomplished by a number of physical, chemical and biological means, all of which can be related to two basic concepts -- storage and localized treatment. Storage is defined as the capturing and containment of CSO's so they can be released to the sewerage system for transport to centralized treatment facilities following the wet weather event. Localized treatment consists of any process or combination of processes that remove pollutants from CSO's to levels considered acceptable for discharge to receiving waters during the wet weather event. Accordingly, CSO control measures can consist of storage, treatment or a combination of both. In this analysis, the unit process of treatment was hydraulically simulated by increasing the conveyance capacity of the sewerage system. During Phase 2 screening, the purpose of structural alternatives was to physically reduce the volume and frequency of CSO discharges on the near-shore waters of Dorchester Bay in order to reduce the high coliform levels present in the water column during and immediately after a CSO event. This short-term coliform impact can be reduced by increasing system storage and/or system conveyance capacity.

In order to determine the most cost-effective and environmentally sound combination of storage and treatment for the Dorchester Bay study area, several alternative combinations were evaluated on the basis of costs, receiving water impacts, and impacts to the physical, biological and social environments. The screening process allows for the evaluation of several levels of storage and/or treatment by utilization of the model SEMSTORM. As discussed in Chapters VII and IX, storage volume and the sewerage system's flow conveyance capacity, (i.e., the simulated treatment) were the variables used in the model to indicate various storage/treatment alternatives. Except for screening, treatment unit processes for the reduction of pollutants were not evaluated during Phase 2.

Phase 2 screening of structural alternatives was performed in the following three steps, and resulted in the selection of alternatives for detailed evaluation:

1. Preliminary Storage - Treatment Analysis: A conceptual technique used to define the total range of storage - treatment alternatives considered for further analysis.

2. Preliminary Selection of Storage - Treatment Alternatives: The development of storage - treatment concepts, the selection of preliminary alternatives, and the determination of receiving water impacts of these alternatives.

3. Screening of Storage - Treatment Alternatives: Estimation of the costs and benefits for each alternative, and the selection of alternatives for detailed evaluation.

Long-term water quality impacts performed during Steps 1 and 2, short-term impacts primarily on the study area's beaches, and estimated costs, all formed the basis for the selection of the screening alternatives that will be evaluated in detail (Chapter XI). The effects on the physical, biological and social environments were also evaluated and considered, as presented in Volume II - Chapters IV and V.

Preliminary Storage - Treatment Analysis. The first step of screening alternative CSO control measures was a conceptual analysis to define the alternatives to be considered in the study, including preliminary, order-of-magnitude cost estimates. This analysis also served to define the range of performance that could be expected from these "boundary condition" alternatives. The S-T combinations used to define the range of potential CSO abatement alternatives were:

- Existing S (Se) and Existing T (Te)
- Existing S (Se) and Maximum T (Tmax)
- Maximum S (Smax) and Existing T(Te)
- Maximum S(Smax) and Maximum T(Tmax)

A definition of each of the above storage - treatment levels follows:

- o Existing S (Se) - estimated existing in-system storage capacity (i.e, 0.22 mil.gal. for Dorchester and 0.48 mil.gal. for South Boston) as described in Chapter IX.

- o Existing T (Te) - estimated existing system conveyance capacity (i.e., 21.0 mgd for Dorchester and 3.3 mgd for South Boston) as described in Chapter IX, assuming the interceptors are clean, and without modifications to the Hoyt Street overflow weir on the Dorchester interceptor (to be discussed later in this chapter)

- o Maximum S (Smax) - defined during the screening process as the storage volume required to capture CSO's generated from a 1-year, 6-hour storm. Smax was estimated for each of the SEMSTORM model sub-areas (described in Chapter IX) by developing hydrographs for each sub-area using the 1-year, 6-hour storm hyetograph and sub-area drainage characteristics; adding to this the average dry weather flow estimated for each sub-area; and then subtracting the sub-area's existing system conveyance capacity (Te). The remaining volume under the hydrograph is the amount of storage (Smax) required to capture the 1-year, 6-hour storm. For each of the three major SEMSTORM sub-areas, Smax values utilized were:

| | |
|---|---|
| A1 (South Boston) | 4.24 mil. gal. |
| A2 (Middle area) | 2.86 mil. gal. |
| A3 (Dorchester) | 14.60 mil. gal. |
| Total | 21.70 mil. gal. |

- o Maximum T (Tmax) - defined during screening as the maximum potential system conveyance capacity based on a reasonable estimate of ultimate existing system expansion. In order to assure that this maximum boundary condition represents a reasonable and feasible expansion of the existing system, it was assumed, during the preliminary S-T analysis, that the Deep Tunnel Plan, as presented in the 1967 report entitled "Improvements to Main Drainage System" by Camp Dresser & McKee Inc. would be adopted. Since this plan was designed to capture a 15-year frequency storm, it was reasonable to assume that it would represent the absolute maximum solution to the CSO problem. In later steps of the screening process, it was determined by the Inner Harbor study area consultant that the Deep Tunnel plan is not a cost-effective solution to the CSO problem (see Inner Harbor Facilities Plan). However, for the purposes of establishing a maximum boundary condition, the use of the Deep Tunnel Plan during this phase of the screening process was considered appropriate. In addition to the Deep Tunnel plan, it was also assumed that additional conveyance capacity would be available to the Dorchester Bay study area at the Columbia Park Headworks and the Calf Pasture Pumping

Station, up to their respective existing maximum hydraulic capabilities. Accordingly, the maximum potential system conveyance capacity for the Dorchester Bay study area was estimated by taking an area-weighted portion of the total hydraulic capacities of each of these facilities. This analysis resulted in an estimated Tmax in the order of 500 mgd, allocated to each of the three major SEMSTORM sub-area's on an area-weighted basis as follows:

| | |
|---|---|
| A1 (South Boston) | 68.6 mgd |
| A2 (Middle area) | 49.3 mgd |
| A3 (Dorchester) | 382.1 mgd |
| Total | 500.0 mgd |

The four boundary S-T combinations were simulated with SEMSTORM over the 29 years of rainfall records. Average CSO frequency, volume and pollutant loadings on an equivalent daily and annual basis were determined for each of the three SEMSTORM sub-areas. The results of these initial boundary condition screening runs are summarized in Table X-3, and are shown graphically on Figures X-1, X-2, and X-3. These figures define the region of possible solutions for CSO abatement, and serve as a graphical aid for selecting additional S-T combinations to be analyzed during the next phase of the screening process.

Simulation results indicate that under existing system operating conditions and hydraulic capacity, CSO's occur on an average about 55 days per year and result in an average annual CSO volume of about 336.2 mil. gal. By providing a total of 21.7 mil. gal. of additional storage volume (Smax), the frequency of CSO's would be reduced to an average of about 8 days per year, and the average annual volume reduced to about 50.4 mil. gal. Providing the maximum potential system conveyance capacity (i.e., maximum treatment rate) of 500 mgd (Tmax) with existing system storage (Se) results in an average of only about 0.2 overflow days per year, producing an average annual volume of about 0.8 mil. gal. The maximum S-T combination (Tmax, Smax) resulted in no CSO events occurring over the 29-year simulation period.

Capital costs for conceptualized facilities associated with the four boundary condition alternatives were estimated in order to develop a range of costs that would represent the spectrum of possible solutions. During this phase of the analysis, only order-of-magnitude present day costs (ENR Index 2900) were developed, primarily for the purpose of estimating potential costs associated with other levels of CSO abatement that fall within the range of possible solutions. The estimated capital cost of Smax is in the order of $18 million; and was approximated by estimating the cost of storage facilities and conveyance conduits required to capture the overflow volume generated from the 1-year, 6-hour storm for each of the SEMSTORM sub-areas (total volume = 21.70 mil. gal.) The estimated capital cost for Tmax is in the order of $330 million, and was estimated by updating the capital cost for the Deep Tunnel Plan as presented in the 1967 report, and assuming that the portion allocated to the

TABLE X-3 RESULTS OF INITIAL SEMSTORM RUNS FOR DETERMINATION OF STORAGE-TREATMENT BOUNDARY CONDITIONS

| SEMSTORM Sub-Area | Alternative | Treatment Rate, MGD | Storage Volume, MIL. GAL. | Overflow Days Per Year | Ave.Vol.Per Overflow Day, MIL.GAL. | Ave. Vol. Per Year, MIL. GAL. | Ave. TSS. Per Year, LBS. | Ave.$BOD_5$ Per Year, LBS. | Ave.Total Colif. Per Yr, #x$10^{12}$ |
|---|---|---|---|---|---|---|---|---|---|
| A1 | | 1.91 | 0.16 | 62.4 | 1.18 | 73.7 | 86,100 | 46,700 | 15,500 |
| A2 | Te-Se | 1.31 | 0.32 | 49.8 | 0.84 | 41.8 | 44,500 | 24,300 | 8,400 |
| A3 | | 21.06 | 0.22 | 53.1 | 4.16 | 220.7 | 253,000 | 137,000 | 46,300 |
| Total | | 24.28 | 0.70 | | 6.18 | 336.2 | 383,600 | 208,000 | 70,200 |
| A1 | | 1.91 | 4.24 | 10.9 | 1.49 | 16.2 | 11,200 | 6,800 | 3,100 |
| A2 | Te-Smax | 1.31 | 2.87 | 9.5 | 1.02 | 9.6 | 6,600 | 4,000 | 1,800 |
| A3 | | 21.06 | 14.60 | 3.4 | 7.27 | 24.6 | 14,900 | 8,800 | 4,200 |
| Total | | 24.28 | 21.71 | | 9.78 | 50.4 | 32,700 | 19,600 | 9,100 |
| A1 | | 68.6 | 0.16 | 0.3 | 1.00 | 0.28 | 200 | 90 | 30 |
| A2 | Tmax-Se | 49.3 | 0.32 | 0.2 | 0.49 | 0.10 | 90 | 40 | 10 |
| A3 | | 382.1 | 0.22 | 0.2 | 2.01 | 0.42 | 400 | 180 | 40 |
| Total | | 500.0 | 0.70 | | 3.50 | 0.80 | 690 | 310 | 80 |
| A1 | | 68.6 | 4.24 | 0 | 0 | 0 | 0 | 0 | 0 |
| A2 | Tmax-Smax | 49.3 | 2.87 | 0 | 0 | 0 | 0 | 0 | 0 |
| A3 | | 382.1 | 14.60 | 0 | 0 | 0 | 0 | 0 | 0 |
| Total | | 500.1 | 21.71 | | 0 | 0 | 0 | 0 | 0 |

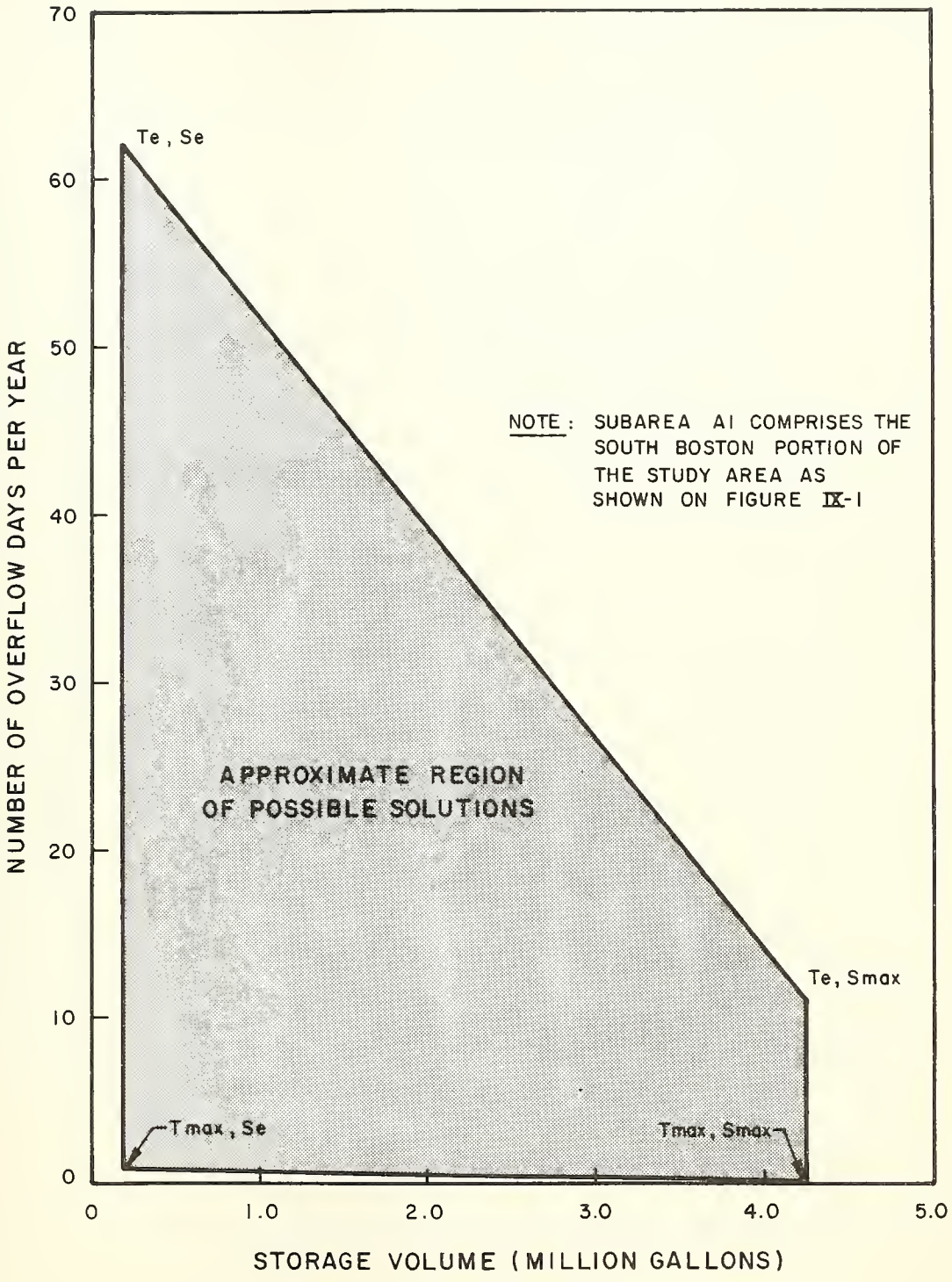


FIG. X-1 LIMITS OF POTENTIAL STORAGE - TREATMENT ALTERNATIVES FOR SUBAREA AI

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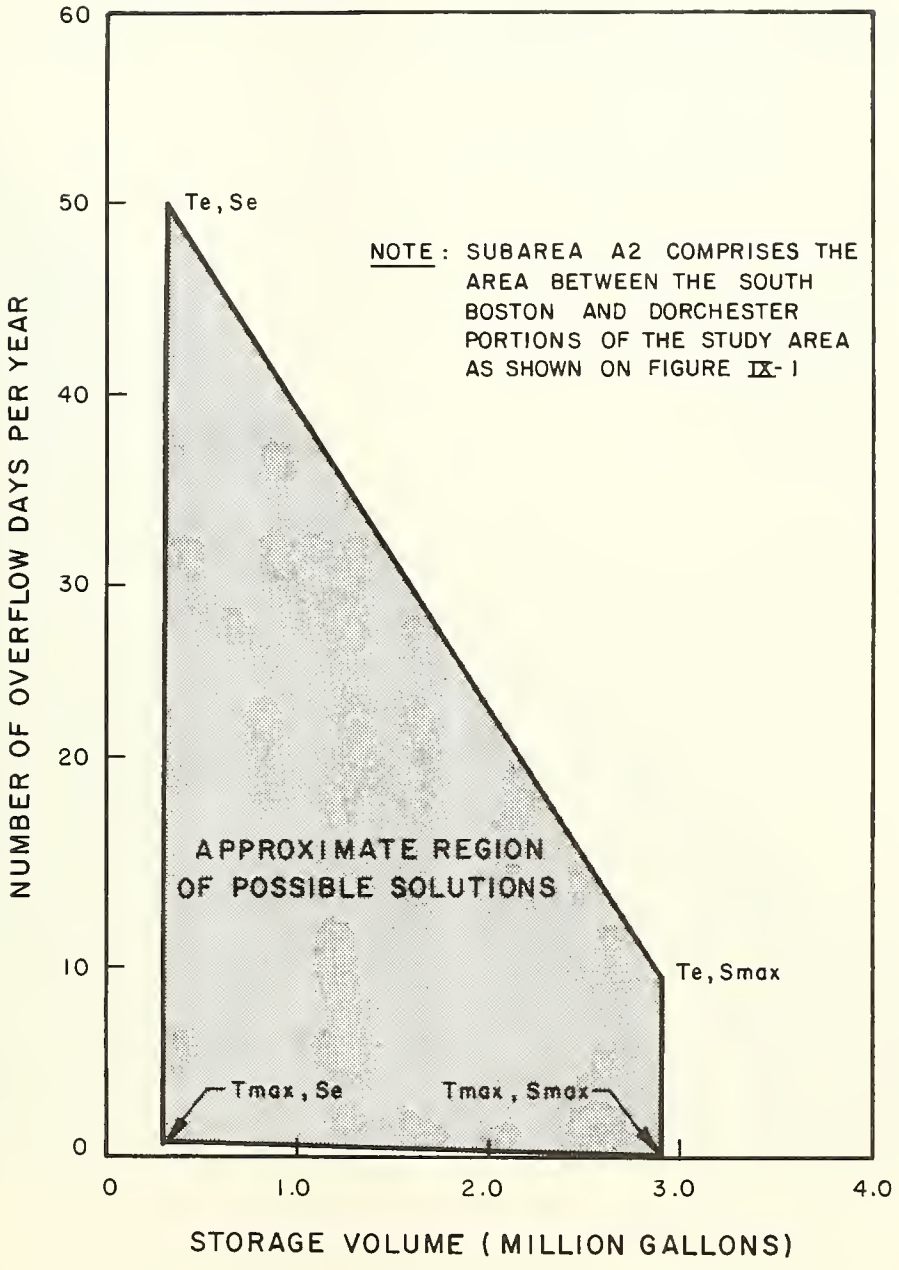


FIG. X-2 LIMITS OF POTENTIAL STORAGE-TREATMENT ALTERNATIVES FOR SUBAREA A2

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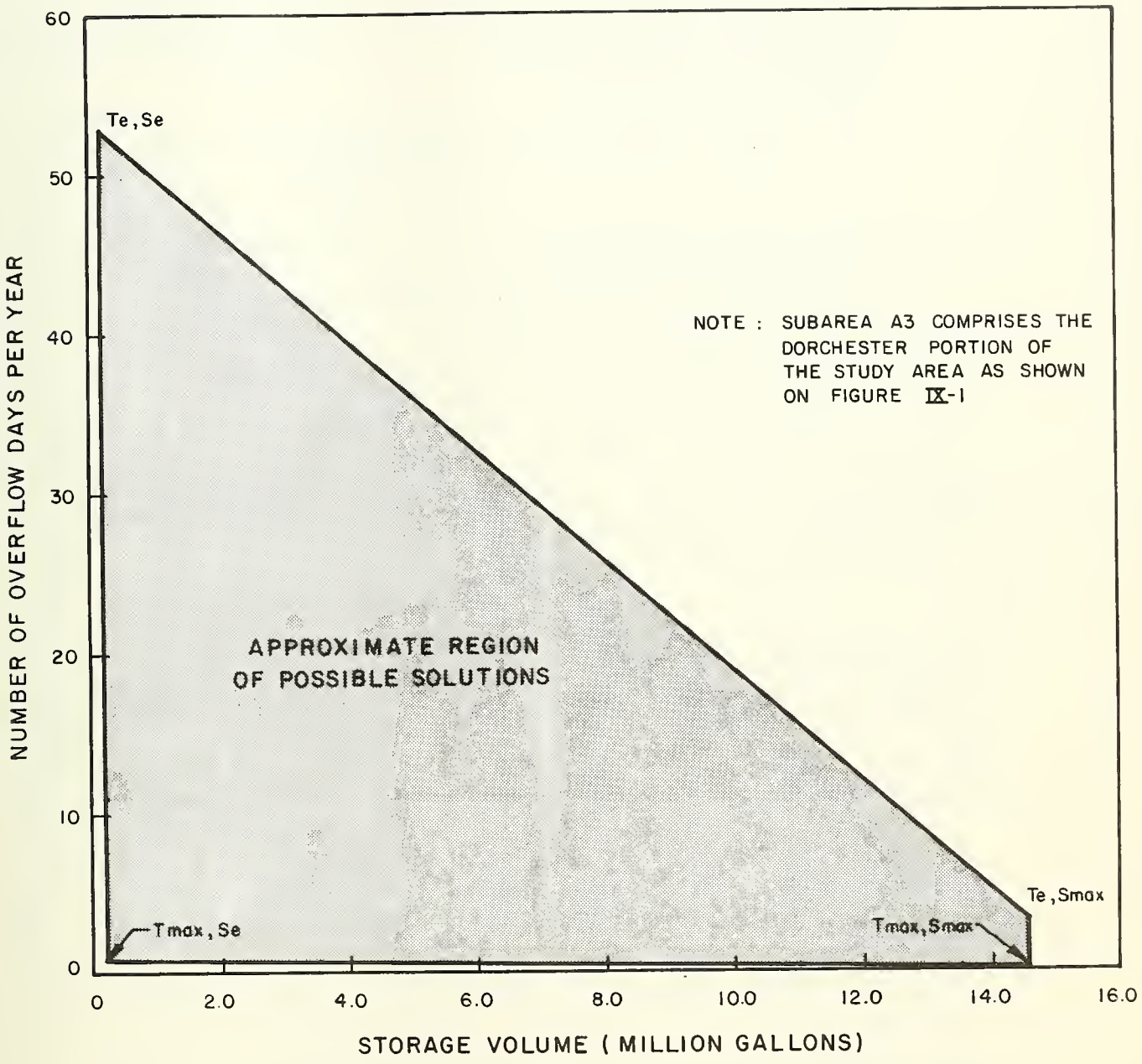


FIG. X-3 LIMITS OF POTENTIAL STORAGE - TREATMENT ALTERNATIVES FOR SUBAREA A3

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Dorchester Bay study area would be in the same proportion as their respective service areas. This cost was added to the Dorchester Bay area portion of the estimated capital costs of upgrading the Calf Pasture/Moon Island facilities, from updating the 1975 EMMA study Alternative 3 costs (Modified Moon Island Plan). A summary of the estimated, order-of-magnitude capital costs for the four boundary condition alternatives is as follows:

| | |
|---|---|
| Te-Se | $0 (existing and future no-action condition) |
| Te-Smax | $18 million |
| Tmax-Se | $330 million |
| Tmax-Smax | $348 million |

Results of the SEMSTORM model runs for the Te-Smax and Tmax-Se conditions were submitted to the HMC in the form of exceedance curves for the purpose of estimating long-term receiving water impacts. The Te-Se condition, which respresents the existing and future no-action condition, was submitted to the HMC earlier and the results are simmarized in Chapter IX. Since the Tmax-Smax condition resulted in no CSO events over the 29-year simulation period there were no impacts to the receiving waters, and thus, no HMC analysis for this condition was necessary. This condition is equivalent to the removal of all DWO's and CSO's as discussed in Chapter IX, with calculated total coliform results at the study area beaches shown on Table IX-4. Similarly, since the impacts of the Smax-Se condition were relatively minimal, the HMC judged that a harbor model run for this condition was not warranted. Thus, harbor water quality impacts were estimated for the Te-Se and Te-Smax conditions only.

Table X-4 shows the results of the harbor impact analysis for calculated total coliform concentrations for the Te-Se and Te-Smax conditions, with and without dry weather overflow (DWO) removed. A comparison of these conditions show that there is no significant long-term coliform reduction realized by providing the storage volume associated with Smax unless DWO is removed. Providing this storage volume, which is equal to the calculated volume of CSO's generated from a 1-year, 6-hour storm (Smax), still results in the contravention of coliform standards at all of the area's beaches except Pleasure Bay, unless DWO is removed. The long-term coliform concentrations at all area beach locations are in compliance with the standards for the Te-Smax condition if DWO is removed. It was demonstrated earlier in this chapter that under existing conditions (Te-Se), the standards are in compliance at all locations except Tenean Beach if DWO is removed, and most likely would be in compliance at Tenean Beach if the Pine Neck Creek storm drain is relocated. Therefore, no structural study area alternatives are specifically required for compliance with long-term coliform standards except possibly for the relocation of the Pine Neck Creek storm drain. It was shown in Chapter IX and earlier in this chapter, that the short-term impacts of total coliform and floatables are the primary receiving water criteria of concern for the Dorchester Bay study area, considering the intensive public use of the near-shore waters and shellfish areas. Accordingly, during the next step of the screening process, storage-treatment concepts that result in a reduction in the frequency of CSO occurrances, and, consequently coliform standard violation days on near-shore areas, were investigated.

TABLE X - 4

HARBOR MODEL CALCULATED TOTAL COLIFORM CONCENTRATIONS
FOR Te-Se AND Te-Smax CONDITIONS[1]

| Location | Harbor Segment Number | Total Coliform Standard | Te-Se | Te-Se DWO Removed[2] | Te-Smax | Te-Smax DWO Removed[2] | All CSO, Removed[3] | All CSO, DWO Removed |
|---|---|---|---|---|---|---|---|---|
| | | CALCULATED 50th PERCENTILE CONCENTRATIONS ($C_{50}$)[4] | | | | | | |
| Pleasure Bay | 29 | 700 | 100 | <10 | 100 | <10 | 100 | <10 |
| Carson Beach | 24 | 700 | 600 | 70 | 500 | 30 | 500 | 30 |
| "L" Street Beach | 26 | 700 | 700 | 80 | 500 | 20 | 500 | 20 |
| Malibu Beach | 12 | 700 | 1,700 | 100 | 1,300 | 40 | 1,300 | 40 |
| Tenean Beach | 213 | 700 | 5,200 | 400 | 4,300 | 100 | 4,300 | 100 |
| | | CALCULATED 80th PERCENTILE CONCENTRATIONS ($C_{80}$)[4] | | | | | | |
| Pleasure Bay | 29 | 1,000 | 300 | <10 | 300 | <10 | 300 | <10 |
| Carson Beach | 24 | 1,000 | 1,700 | 500 | 1,300 | 200 | 1,200 | 200 |
| "L" Street Beach | 26 | 1,000 | 1,800 | 500 | 1,400 | 100 | 1,300 | 100 |
| Malibu Beach | 12 | 1,000 | 4,500 | 800 | 3,600 | 200 | 3,600 | 200 |
| Tenean Beach | 213 | 1,000 | 14,300 | 2,200 | 12,000 | 800 | 11,900 | 700 |



Notes

[1]From Tables 2a and 2b of Hydroscience's May 3, 1979 "Boston Combined Sewer Overflow Study - Harbor Model Parameters for the Analysis of Total Coliform, Dissolved Oxygen and Total Suspended Solids - Te Se, Te Smax"- see Chapter IX.

[2]With existing known project area (total) dry weather overflows (DWO) eliminated, total coliform calculated at the location during wet weather period.

[3]With project area (total) CSO's eliminated, total coliform calculated at the location during wet weather period.

[4]MPN per 100 ml.

Preliminary Selection of Storage - Treatment Alternatives. The purpose of this step of the screening process is to select various alternative storage-treatment combinations that fall within the region of possible solutions, and to estimate for each alternative the resulting impacts on the receiving waters. In order to simplify the selection process, the storage-treatment curves for SEMSTORM sub-areas A1 and A2 (Figures X-1 and X-2) were combined and a new curve developed representing the entire South Boston portion of the study area. Since these two sub-areas are hydraulically connected, and the CSO outlets from both areas are in relative close proximity to each other and discharge to the same water body (Old Harbor), it appears reasonable at this stage of the analysis to assume that potential structural alternatives would involve consolidation of these outfalls, with storage and/or treatment at one or possibly two locations in South Boston. A preliminary, order-of-magnitude cost analysis comparing storage at several outfall locations in South Boston to storage at one or two locations using the Smax volume substantiated this conclusion.

Figures X-4 and X-5 show the number of overflow days per year vs. storage-treatment curves for the Dorchester and South Boston portions of the study area, respectively. Also shown are additional storage-treatment curves generated by selecting various treatment rates between Te and Tmax, and storage volumes between Se and Smax, and simulating these new combinations over the 29-year period with SEMSTORM. The average number of overflow days per year was determined from model output for each combination.

Some of the selected treatment rates are based on project-oriented concepts that appear to have potential. For example, referring to Figure X-4, the treatment rates Te, T1, T2 and T3 are based on various flow conditions in the Dorchester interceptor with Te representing the nominal flowing full capacity, and T1, T2 and T3 representing various levels of surcharge. By modifying the side overflow weir regulator located on the interceptor near Hoyt Street (see Figure C, appended), the interceptor can operate under surcharged conditions without resulting in an overflow at this location. It should be noted, as discussed in Chapter IX, that these treatment rates are based on the Dorchester area portion of the interceptor's capacity only, since the Neponset study area is also tributary to this conduit. The existing Dorchester treatment rate of 21.0 mgd (Te) is based on the interceptor's nominal, flowing full capacity of 28.2 mgd, and treatment rate T3 ( 48.0 mgd) is based on the calculated maximum surcharge capacity of 60.0 mgd. The treatment rates T4, T5, and T6 were selected at random and reflect additional system conveyance capacity.

South Boston treatment rates (Figure X-5) were selected in a similar manner, with Te being the current average, and T1 the potential maximum, area-weighted proportioned flowrates at the Columbus Park Headworks. Treatment rates T2 through T5 were selected at random and reflect additional system conveyance capacity.

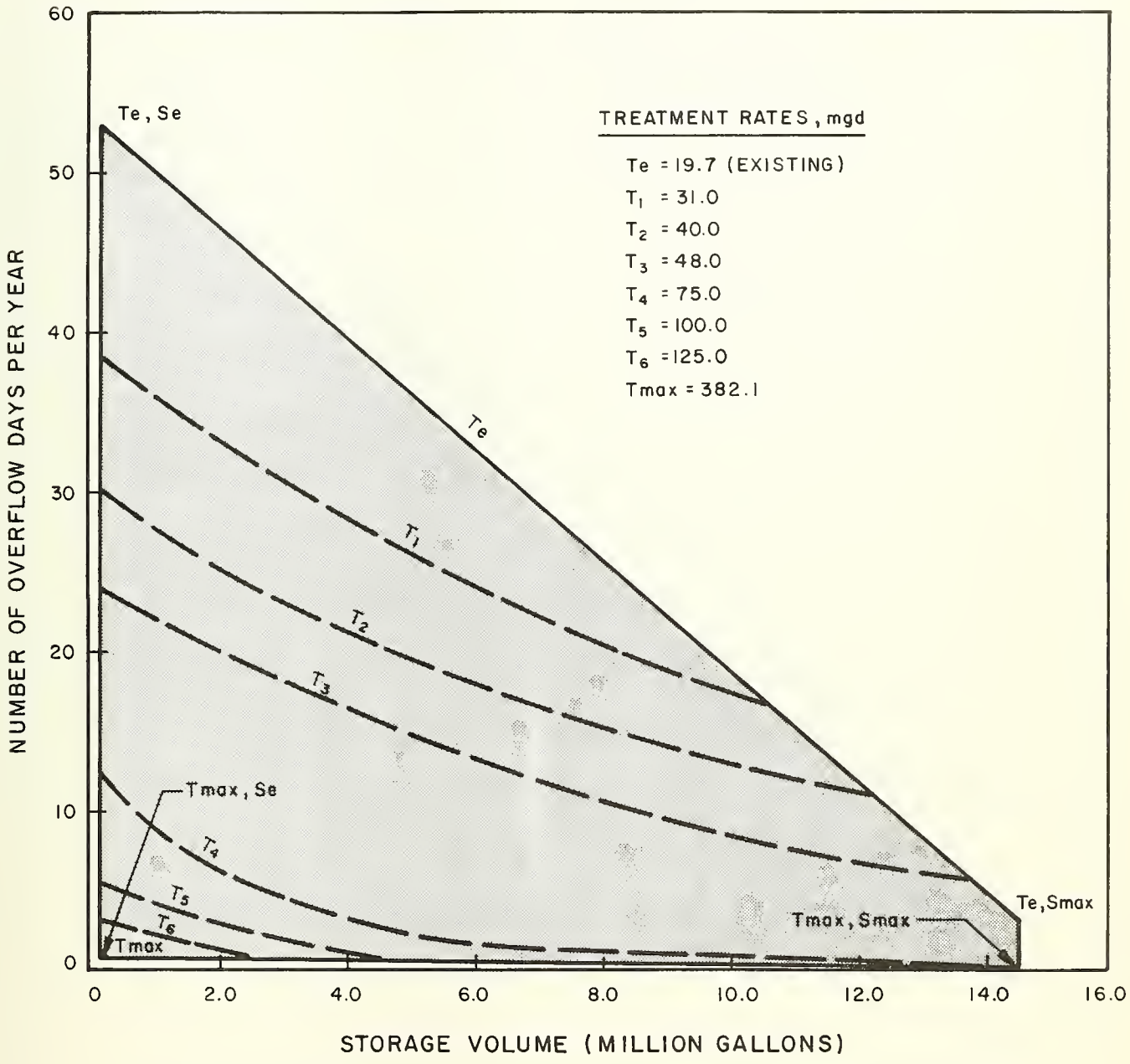


FIG. X-4 OVERFLOW DAYS VS. STORAGE - TREATMENT CURVES FOR DORCHESTER



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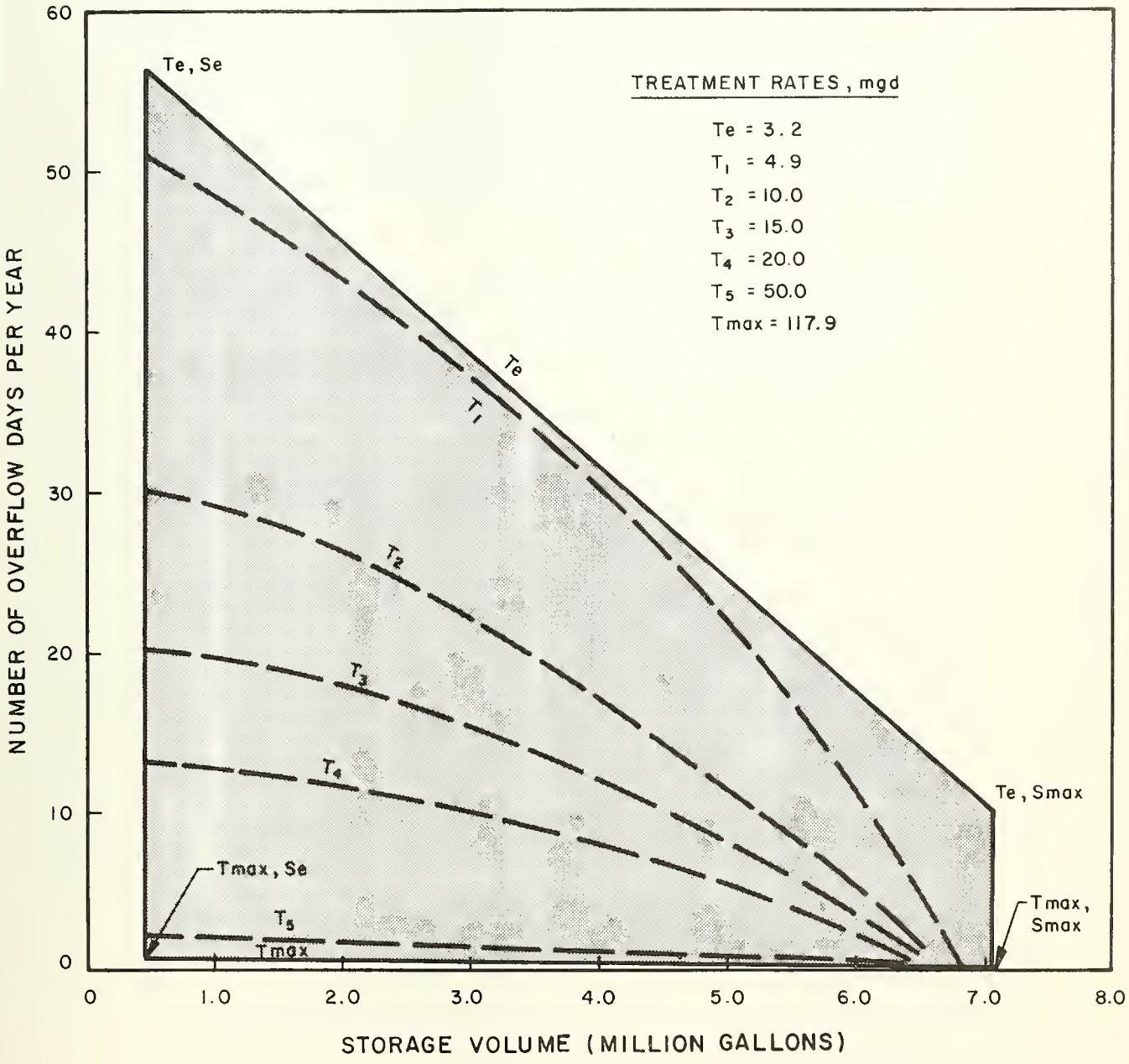


FIG. X-5 OVERFLOW DAYS VS. STORAGE-TREATMENT CURVES FOR SOUTH BOSTON

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Based on the study area issues and information generated during the storage-treatment analysis discussed above, three conceptual study area alternatives were developed that would result in a reduction in floatables, and in the frequency of CSO occurrances. The alternatives were based on the following two concepts:

Storage/containment - where the existing overflows would be collected, screened, stored during the wet weather period, and then bled back into the interceptor system when conveyance capacity becomes available. CSO volumes exceeding the storage capacity would overflow via the existing outfalls and discharge to Dorchester Bay; however, the overflows would be screened for the removal of floatables.

Extended Discharges of CSO's - where the existing overflows would be collected by consolidation conduits, screened, and discharged (either by gravity or pumping) away from the bathing and shellfish areas to outer locations that have a greater assimulative capacity than the relatively shallow existing CSO outlet locations. It should be noted that this concept does not result in a reduction in the total number of overflow days per year (since overflows would occur at the selected further offshore location(s)), but in a reduction in the number of overflow days and consequently coliform violation days, at near-shore bathing and shellfish areas. When the capacity of the overflow outlet extension facilities is exceeded, an overflow would occur via the existing outlets into the near-shore areas.

Three alternatives were developed in order to estimate the long-term total coliform impacts on the near-shore waters of Dorchester Bay with the implementation of the above two concepts. Primary emphasis was centered on Tenean Beach where there may be a contravention of the coliform standards even with DWO removed and Pine Neck Creek storm drain relocated. Results of the HMC analysis of these alternative concepts will aid in the selection of the final storage-treatment alternatives to be evaluated during the final step of the screening process. The three alternatives submitted to the HMC for harbor impact analysis were as follows:

| Alternative | South Boston | Dorchester |
|---|---|---|
| 1 | Consolidation of CSO's and discharge to Inner Harbor for Te-Se condition | Consolidation of CSO's and discharge to the outer area of Dorchester Bay under T1-Se condition |
| 2 | Consolidation of CSO's and discharge to outer Dorchester Bay under Te-Se condition | Provide 1.0 mil. gal. of off-line storage; treatment rate = T1 |
| 3 | Consolidation of CSO's and discharge to outer Dorchester Bay under Te-Se condition | Provide 3.1 mil. gal. of off-line storage; treatment rate = T2 |

Since the long-term total coliform levels at all South Boston beaches are in compliance with the SB standards under existing conditions with DWO removed, alternatives involving the discharging of South Boston CSO volumes to outer waters of Dorchester Bay and Inner Harbor were selected to primarily determine the impact on the remaining sections of the bay. These were combined with alternatives involving nominal levels of storage and treatment for the Dorchester area.

Results of the HMC analysis of the three alternatives are shown in Table X-5. The following conclusions can be made based on the results:

- o A nominal reduction in long-term total coliform concentrations at all beach locations is evident, but the total coliform standard is still not met at Tenean Beach.

- o Since the extent of contravention at Tenean Beach is relatively small (200 MPN per 100 ml), it can be concluded that the relocation of the Pine Neck Creek storm drain, in conjunction with any of these alternatives, would result in compliance with the total coliform standards.

- o If the South Boston CSO's were to be consolidated and discharged to the Inner Harbor (Alternative 1) or outer Dorchester Bay (Alternatives 2 and 3), the location of the discharge does not impact substantially on the long-term total coliform concentrations at the study area's beaches.

- o As discussed previously, at all locations except Tenean Beach, the long-term total coliform standards are met in Dorchester Bay.

Screening Storage-Treatment Alternatives. This final step of the screening process includes the development of the alternatives for final screening, and an evaluation of the costs and benefits associated with each. Alternatives were selected that resulted in the reduction of the frequency of CSO occurrances, and consequently, short-term coliform impacts on the near-shore areas of Dorchester Bay. Removal of floatables was also included as part of each alternative screened. Alternatives were based on (1) storage/containment, and (2) extended discharges of CSO's.

Three alternatives were developed for the Dorchester portion of the study area. Also, an alternative considered in a previous study(1) involving the protection of the area's only two beaches (Malibu and Tenean) utilizing diked enclosures was investigated. This alternative will be discussed in a subsequent section of the chapter. All three Dorchester area alternatives include the relocation of the Pine Neck Creek storm drain. This would involve an extension of the conduit southeasterly to route stormwater discharges to the Neponset River. A possible route for the extension is shown on Figure X-II, presented later in this chapter.

(1) Camp Dresser & McKee Inc., "Water Quality Improvement of Tenean and Malibu Beaches," for Massachusetts Division of Water Pollution Control, November 1972.

TABLE X - 5 HARBOR MODEL CALCULATED TOTAL COLIFORM CONCENTRATIONS FOR PRELIMINARY STORAGE-TREATMENT ALTERNATIVES[1]

| Location | Harbor Segment Number | Total Coliform Standard | Te-Se DWO Removed[2] | Dorchester Bay Area Alternatives[2] 1 | 2 | 3 |
|---|---|---|---|---|---|---|
| | CALCULATED 50th PERCENTILE CONCENTRATIONS ($C_{50}$)[3] | | | | | |
| Pleasure Bay | 29 | 700 | < 10 | 10 | 10 | 10 |
| Carson Beach | 24 | 700 | 70 | 50 | 60 | 60 |
| "L" Street Beach | 26 | 700 | 80 | 20 | 60 | 50 |
| Malibu Beach | 12 | 700 | 100 | 80 | 90 | 70 |
| Tenean Beach | 213 | 700 | 400 | 230 | 230 | 220 |
| | CALCULATED 80th PERCENTILE CONCENTRATIONS ($C_{80}$)[3] | | | | | |
| Pleasure Bay | 29 | 1,000 | < 10 | 15 | 20 | 15 |
| Carson Beach | 24 | 1,000 | 500 | 260 | 450 | 360 |
| "L" Street Beach | 26 | 1,000 | 500 | 120 | 340 | 300 |
| Malibu Beach | 12 | 1,000 | 800 | 420 | 470 | 380 |
| Tenean Beach | 213 | 1,000 | 2,200 | 1,200 | 1,200 | 1,200 |

Notes

[1] From Tables 5 and 6 of Hydroscience's July 16, 1979 "Boston Combined Sewer Overflow Study - Harbor Model Results for Evaluation of Alternatives".

[2] With existing known project area (total) dry weather overflows (DWO) eliminated, total coliform calculated at the location during wet weather period.

[3] MPN per 100 ml.

The three Dorchester area alternatives are shown on Figure X-6 and are described below. Under each alternative, CSO's exceeding the capacity of the facilities would be screened and discharged through the existing outlets.

| Alternatives | Description |
|---|---|
| D-1 | For CSO outlets BOS-088/089 and BOS-090, install screening and pumping facilities on or adjacent to each overflow conduit and pump CSO's via a 10,000-foot long force main from BOS-090, and a 4500-foot long force main from BOS-089 to outer Dorchester Bay. |
| D-2 | Install two underground storage tanks, including screening facilities, on or adjacent to the two CSO outlet conduits for containment of CSO's. Discharge stored CSO's to the Dorchester Interceptor when conveyance capacity becomes available. |
| D-3 | Extend the CSO conduit tributary to outlet BOS-090 northward to Fox Point where a screening facility would be located serving this outlet and also BOS-088/089. The screened CSO's would discharge to outer Dorchester Bay via a 4500-foot long gravity outfall conduit. |

The facilities associated with each alternative were sized and costs estimated for varying reductions in the number of violation days per year. A violation day is defined as an overflow occurance at or near a study area beach. As shown in Chapter IX, the total coliform standards are contravened immediately after an overflow event (based on an average CSO volume from SEMSTORM), thus a "violation" would result. For example, under Alternatives D-1 and D-3, the number of overflow days per year would not be reduced. However, since the CSO's would be discharged at a location away from the beaches, the number of days per year that the total coliform level at the beaches would exceed the standards (i.e., violation day) would be reduced. Figure X-4 was used to approximate the capacity of required facilities for each violation day per year condition, since storage volume represents required storage tank capacity, and treatment rate is equivalent to conveyance conduit and/or pumping capacity. As mentioned previously, it can be operated at a maximum surcharge capacity of about 60.0 mgd, resulting in a treatment rate for Dorchester of about 48.0 mgd. (The remaining 12.0 mgd is allocated to the Neponset study area). As shown on Figure X-4, this treatment rate (T3) reduces the number of overflow days per year from 53 to 24, or a 55 percent reduction. Accordingly, this regulator modification was included as part of each Dorchester alternative. It would consist of blocking the side overflow weir-type regulator located on the interceptor near Hoyt Street, thus preventing the occurrence of overflows at this location during surcharge conditions.

The facilities associated with each of the three Dorchester alternatives were sized and costs estimated for an allowance of 0, 6, 12, and 24 violation days per year. Table X-6 shows the results of the cost analysis and the resultant percent reductions in overflow days per year and violation days per year at the beaches. Figure X-7 shows a comparison of the estimated total annual cost associated with the corresponding reduction in the number of violation days per year for each alternative. As shown, Alternative D-2 (storage/ containment) is the most cost effective.

TABLE X-6 COMPARISON OF SCREENED STUDY AREA
STRUCTURAL ALTERNATIVES

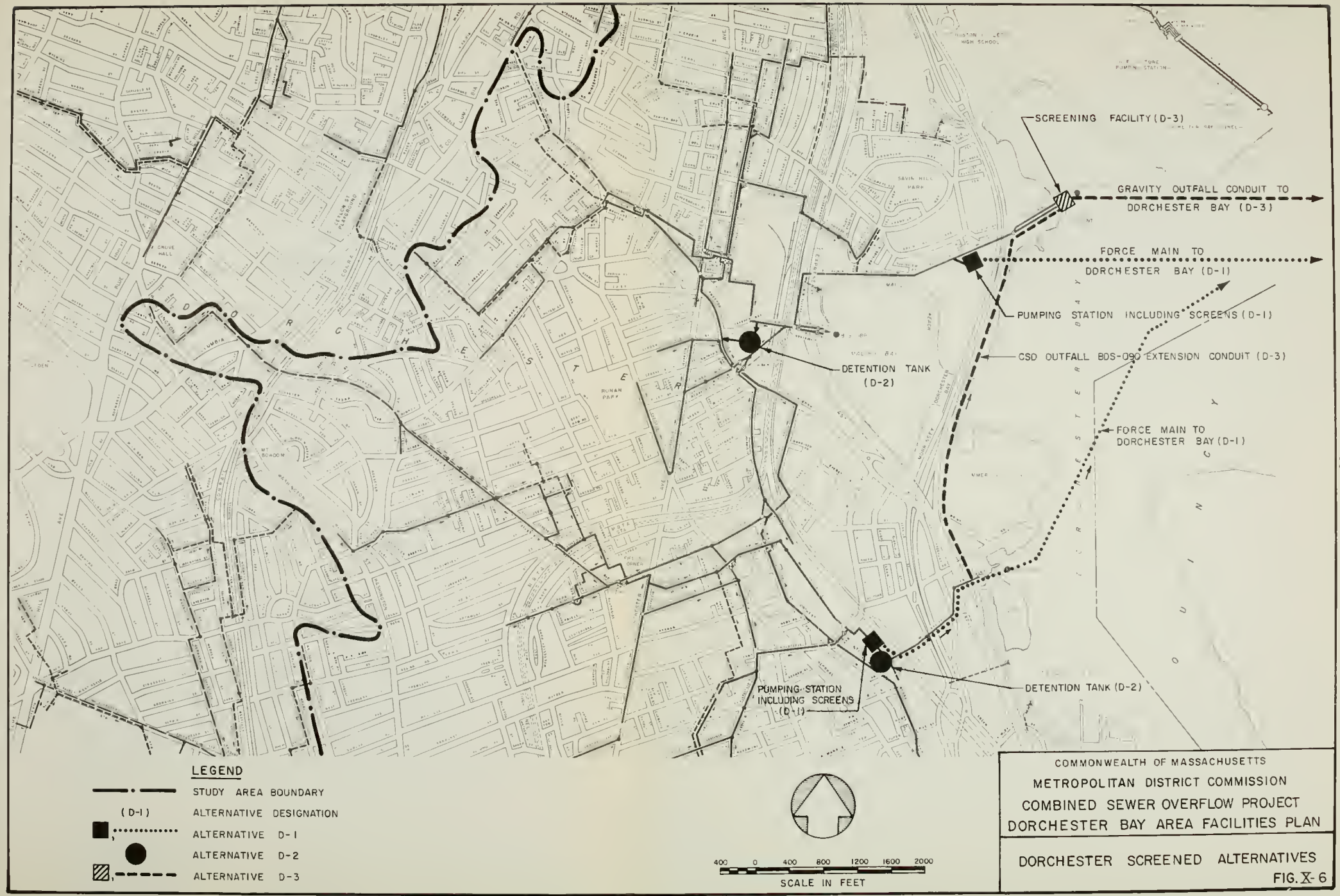
| ALTERNATIVES | | PRELIMINARY ESTIMATED COSTS (MILLION $)[1] | | | | | PERCENT REDUCTION | |
|---|---|---|---|---|---|---|---|---|
| DESIGNATION/DESCRIPTION | VIOLATION DAYS PER YR. | CAPITAL | AMORTIZED CAP.[2] | O & M | TOTAL ANNUAL | PRESENT WORTH | OVERFLOW DAYS PER YR. | VIOLATION DAYS/YR. AT BEACHES |
| DORCHESTER | | | | | | | | |
| D-1 | 0 | 66.9 | 6.26 | 0.20 | 6.46 | 69.1 | 0 | 100 |
| Screen, pump to | 6 | 34.5 | 3.23 | 0.04 | 3.27 | 35.0 | 0 | 88 |
| Dor. Bay | 12 | 19.6 | 1.83 | 0.02 | 1.85 | 19.8 | 0 | 77 |
| | 24 | 0.3 | 0.03 | 0 | 0.03 | 0.3 | 0 | 54 |
| D-2 | 0 | 16.3 | 1.52 | 0.01 | 1.53 | 16.4 | 100 | 100 |
| Storage/Containment, | 6 | 11.5 | 1.08 | 0.01 | 1.09 | 11.7 | 88 | 88 |
| Disch. to collection system | 12 | 8.0 | 0.75 | 0.01 | 0.76 | 8.1 | 77 | 77 |
| | 24 | 0.3 | 0.03 | 0 | 0.03 | 0.3 | 54 | 54 |
| D-3 | 0 | 49.4 | 4.63 | 0.08 | 4.71 | 50.4 | 0 | 100 |
| Consolidate, screen, gravity | 6 | 43.6 | 4.08 | 0.08 | 4.16 | 44.5 | 0 | 88 |
| Disch. to Dor. Bay | 12 | 38.4 | 3.59 | 0.08 | 3.67 | 39.3 | 0 | 77 |
| | 24 | 0.3 | 0.03 | 0 | 0.03 | 0.3 | 0 | 54 |
| BEACH ENCLOSURES[3] | 0 | 13.8 | 1.30 | 0.10 | 1.40 | 15.0 | 0 | 100 |
| Dikes at Malibu and Tenean beaches | | | | | | | | |
| SOUTH BOSTON | | | | | | | | |
| SB-1 | 0 | 28.7 | 2.68 | 0.20 | 2.88 | 30.8 | 0 | 100 |
| Consolidate, screen, pump | 10 | 11.8 | 1.10 | 0.05 | 1.15 | 12.3 | 0 | 82 |
| to Inner Harbor | 20 | 5.5 | 0.51 | 0.03 | 0.54 | 5.8 | 0 | 64 |
| | 40 | 4.3 | 0.40 | 0.01 | 0.41 | 4.4 | 0 | 29 |
| SB-2 | 0 | 22.7 | 2.12 | 0.02 | 2.14 | 22.9 | 0 | 100 |
| Consolidate, screen, gravity | 10 | 13.9 | 1.30 | 0.02 | 1.32 | 14.1 | 0 | 82 |
| discharge to Inner Harbor | 20 | 6.6 | 0.62 | 0.01 | 0.63 | 6.7 | 0 | 64 |
| | 40 | 4.4 | 0.41 | 0.01 | 0.42 | 4.5 | 0 | 29 |
| SB-3 | 0 | 31.8 | 2.97 | 0.19 | 3.16 | 33.8 | 0 | 100 |
| Consolidate, screen, pump | 10 | 13.5 | 1.26 | 0.04 | 1.30 | 13.9 | 0 | 82 |
| to Dor. Bay | 20 | 6.3 | 0.59 | 0.03 | 0.62 | 6.6 | 0 | 64 |
| | 40 | 5.2 | 0.49 | 0.01 | 0.50 | 5.3 | 0 | 29 |
| SB-4 | 0 | 26.8 | 2.51 | 0.02 | 2.53 | 27.1 | 0 | 100 |
| Consolidate, screen, gravity | 10 | 13.2 | 1.23 | 0.02 | 1.25 | 13.4 | 0 | 82 |
| discharge to Dor. Bay | 20 | 8.2 | 0.77 | 0.01 | 0.78 | 8.3 | 0 | 64 |
| | 40 | 5.1 | 0.48 | 0.01 | 0.49 | 5.2 | 0 | 29 |
| SB-5 | 0 | 10.7 | 1.00 | 0.02 | 1.02 | 10.9 | 100 | 100 |
| Consolidate, storage (tank), | 10 | 8.0 | 0.75 | 0.02 | 0.77 | 8.2 | 82 | 82 |
| disch. to collection system | 20 | 6.6 | 0.62 | 0.01 | 0.63 | 6.7 | 64 | 64 |
| | 40 | 4.1 | 0.38 | 0.01 | 0.39 | 4.2 | 29 | 29 |
| SB-6 | 0 | 18.5 | 1.73 | 0.03 | 1.76 | 18.8 | 100 | 100 |
| Consolidate, storage (conduit), | 10 | 15.0 | 1.40 | 0.03 | 1.43 | 15.3 | 82 | 82 |
| disch. to collection system | 20 | 12.6 | 1.18 | 0.02 | 1.20 | 12.8 | 64 | 64 |
| | 40 | 5.3 | 0.50 | 0.01 | 0.51 | 5.5 | 29 | 29 |

NOTES:

(1) Based on an Engineering News Record (ENR) Construction Cost Index of 2900 which represents June, 1979 price levels, - includes 30% allowance for engineering and contingencies.

(2) Based on a discount rate of 6 7/8 percent per annum and assumes equal annual payments over the 20-year planning period (Capital Recovery Factor (CRF) = 0.09348)

(3) This alternative represents protection of bathing areas only, thus there would be no CSO abatement for other near and off-shore areas of Dorchester Bay. See discussion later in this chapter.


SCREENING FACILITY (D-3)
GRAVITY OUTFALL CONDUIT TO DORCHESTER BAY (D-3)
FORCE MAIN TO DORCHESTER BAY (D-1)
PUMPING STATION INCLUDING SCREENS (D-1)
CSO OUTFALL BOS-090 EXTENSION CONDUIT (D-3)
DETENTION TANK (D-2)
FORCE MAIN TO DORCHESTER BAY (D-1)
DETENTION TANK (D-2)
PUMPING STATION INCLUDING SCREENS (D-1)
LEGEND
STUDY AREA BOUNDARY
(D-1) ALTERNATIVE DESIGNATION
ALTERNATIVE D-1
ALTERNATIVE D-2
ALTERNATIVE D-3
400 0 400 800 1200 1600 2000
SCALE IN FEET
COMMONWEALTH OF MASSACHUSETTS
METROPOLITAN DISTRICT COMMISSION
COMBINED SEWER OVERFLOW PROJECT
DORCHESTER BAY AREA FACILITIES PLAN
DORCHESTER SCREENED ALTERNATIVES
FIG. X-6

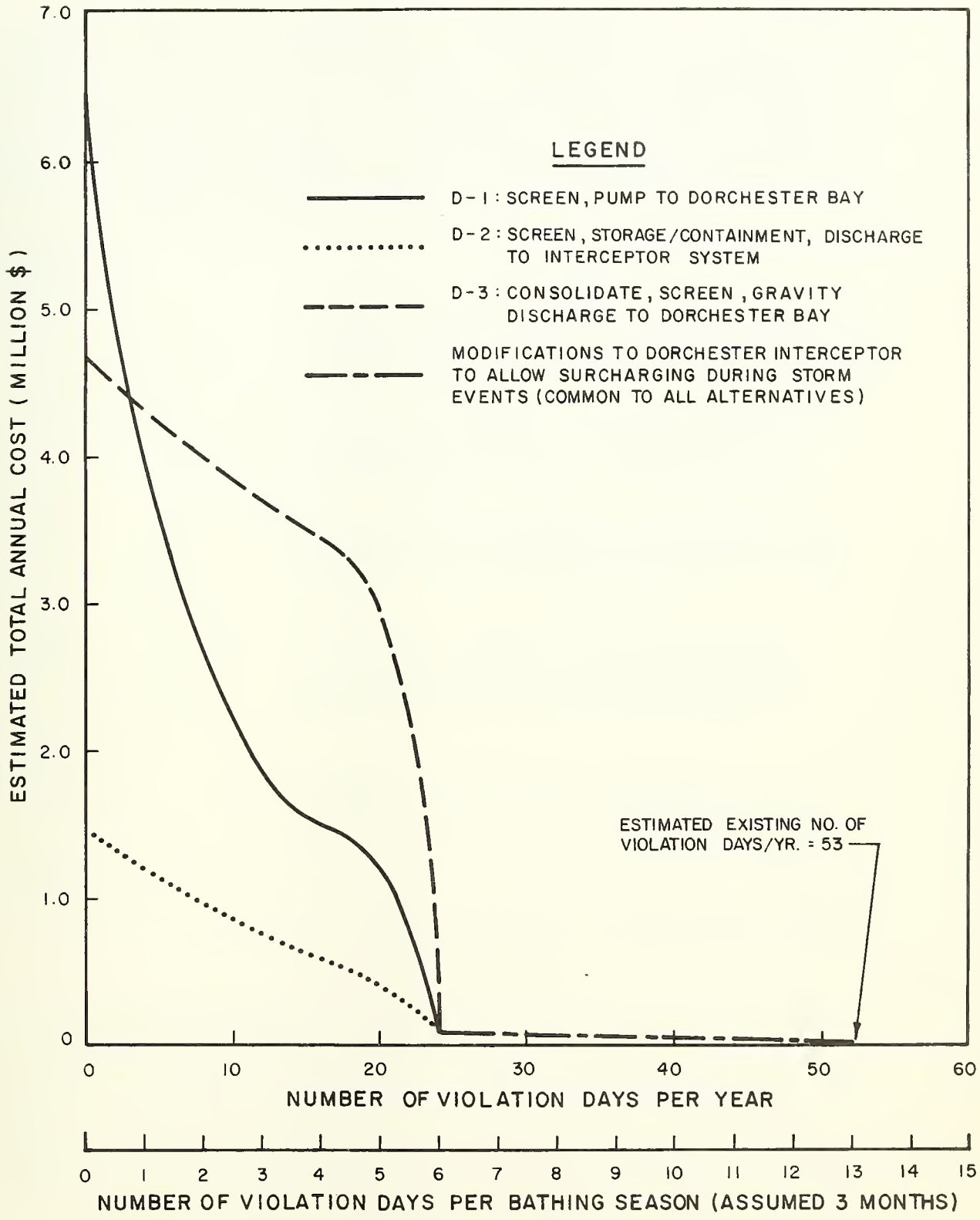

LEGEND
D-1 : SCREEN, PUMP TO DORCHESTER BAY
D-2 : SCREEN, STORAGE/CONTAINMENT, DISCHARGE TO INTERCEPTOR SYSTEM
D-3 : CONSOLIDATE, SCREEN, GRAVITY DISCHARGE TO DORCHESTER BAY
MODIFICATIONS TO DORCHESTER INTERCEPTOR TO ALLOW SURCHARGING DURING STORM EVENTS (COMMON TO ALL ALTERNATIVES)
ESTIMATED EXISTING NO. OF VIOLATION DAYS/YR. = 53
ESTIMATED TOTAL ANNUAL COST ( MILLION $ )
7.0
6.0
5.0
4.0
3.0
2.0
1.0
0
0 10 20 30 40 50 60
NUMBER OF VIOLATION DAYS PER YEAR
0 1 2 3 4 5 6 7 8 9 10 11 12 13 14 15
NUMBER OF VIOLATION DAYS PER BATHING SEASON (ASSUMED 3 MONTHS)

FIG. X-7 VIOLATION DAYS VS. COST FOR DORCHESTER AREA SCREENED ALTERNATIVES

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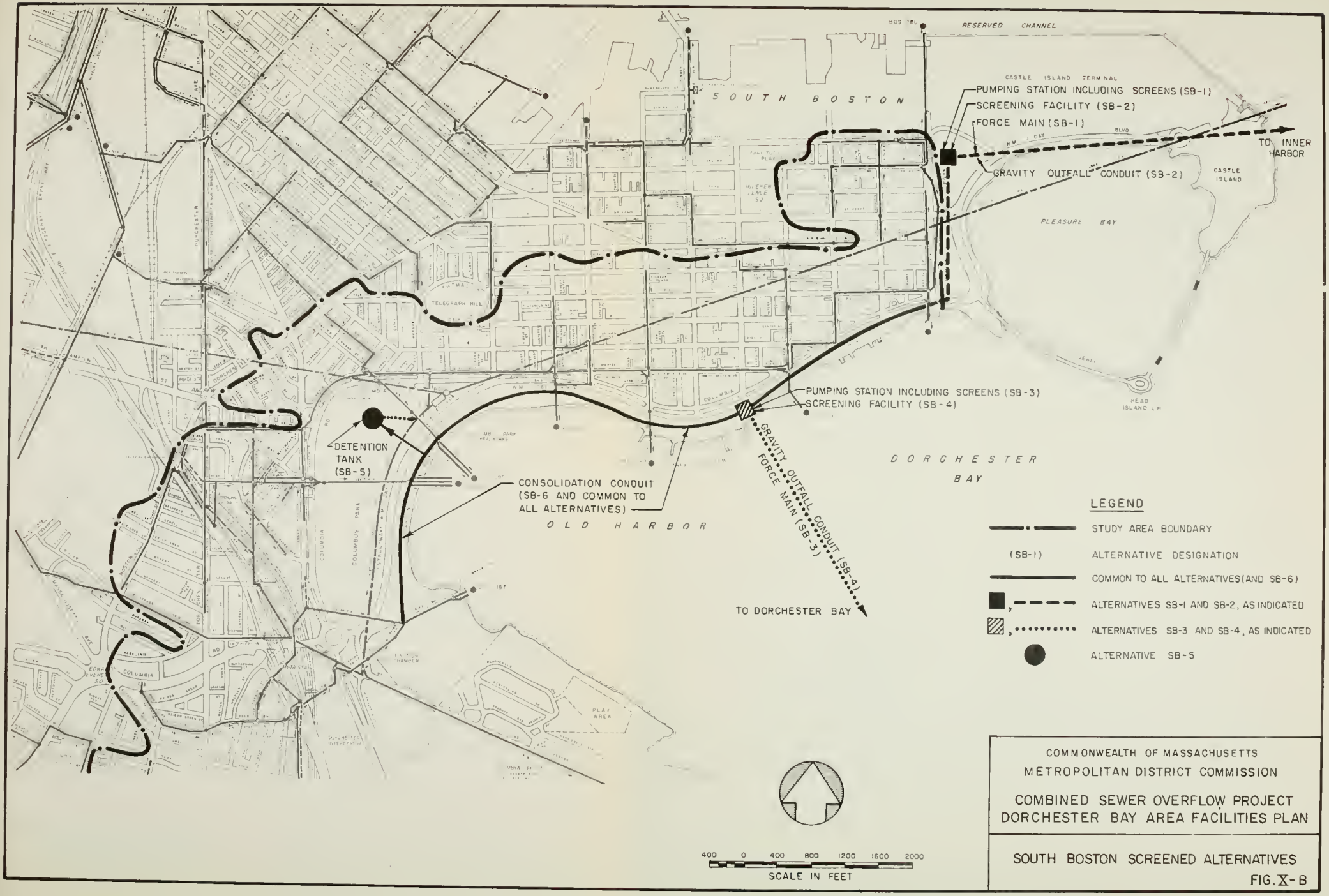
Six alternatives, based on the concepts of storage/containment and outer discharges of CSO's, were developed for the South Boston portion of the study area. All involve consolidation of CSO's tributary to outlets BOS-081 through BOS-087, and locating storage or facilities associated with offshore discharges at a single location. Considering the relatively small tributary area, the nature of the receiving water, and the minor negligible impacts to the harbor, control of CSO's tributary to BOS-080 was not evaluated during the screening phase. The alternatives are shown on Figure X-8 and are described below:

| Alternatives | Description |
|---|---|
| SB-1 | Install a pumping station, equipped with screens, at Marine Park and pump CSO's via a 5,000-foot long force main to Inner Harbor. |
| SB-2 | Install a screening facility at Marine Park and discharge CSO's to Inner Harbor via a 5,000 foot long gravity outfall conduit. |
| SB-3 | Install a pumping station, including screens, at M Street, extended, and pump CSO's via a 5,000 foot long force main to outer Dorchester Bay. |
| SB-4 | Install a screening facility at M Street, extended, and discharge CSO's to outer Dorchester Bay via a 5000-foot long gravity outfall conduit. |
| SB-5 | Install an underground storage tank at Columbus Park for the containment of CSO's, and discharge stored CSO to the Columbus Park Connection when capacity becomes available. |
| SB-6 | Utilize the CSO outlet consolidation conduit for storage/containment, and discharge stored flow to the Columbus Park Headworks when capacity becomes available. |

Similar to the Dorchester area alternatives, the facilities associated with each South Boston area alternative were sized and costs estimated for varying reductions in the number of violation days per year at the study area's beaches. Figure X-5 was utilized to estimate the required capacity of facilities associated with each alternative's overflow day-per-year condition. Since existing (Te, Se) conditions result in an estimated 56 overflow days per year, the facilities associated with each alternative were sized and costs estimated for an allowance of 0, 10, 20, and 40 violation days per year at the area's beaches. Table X-6 and Figure X-9 show the results of the analysis. Results indicate that alternative SB-5 is the most cost-effective alternative for levels of reduction between 0 and 16 violation days per year.

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COMBINED SEWER OVERFLOW PROJECT
DORCHESTER BAY AREA FACILITIES PLAN

SOUTH BOSTON SCREENED ALTERNATIVES
FIG. X-8

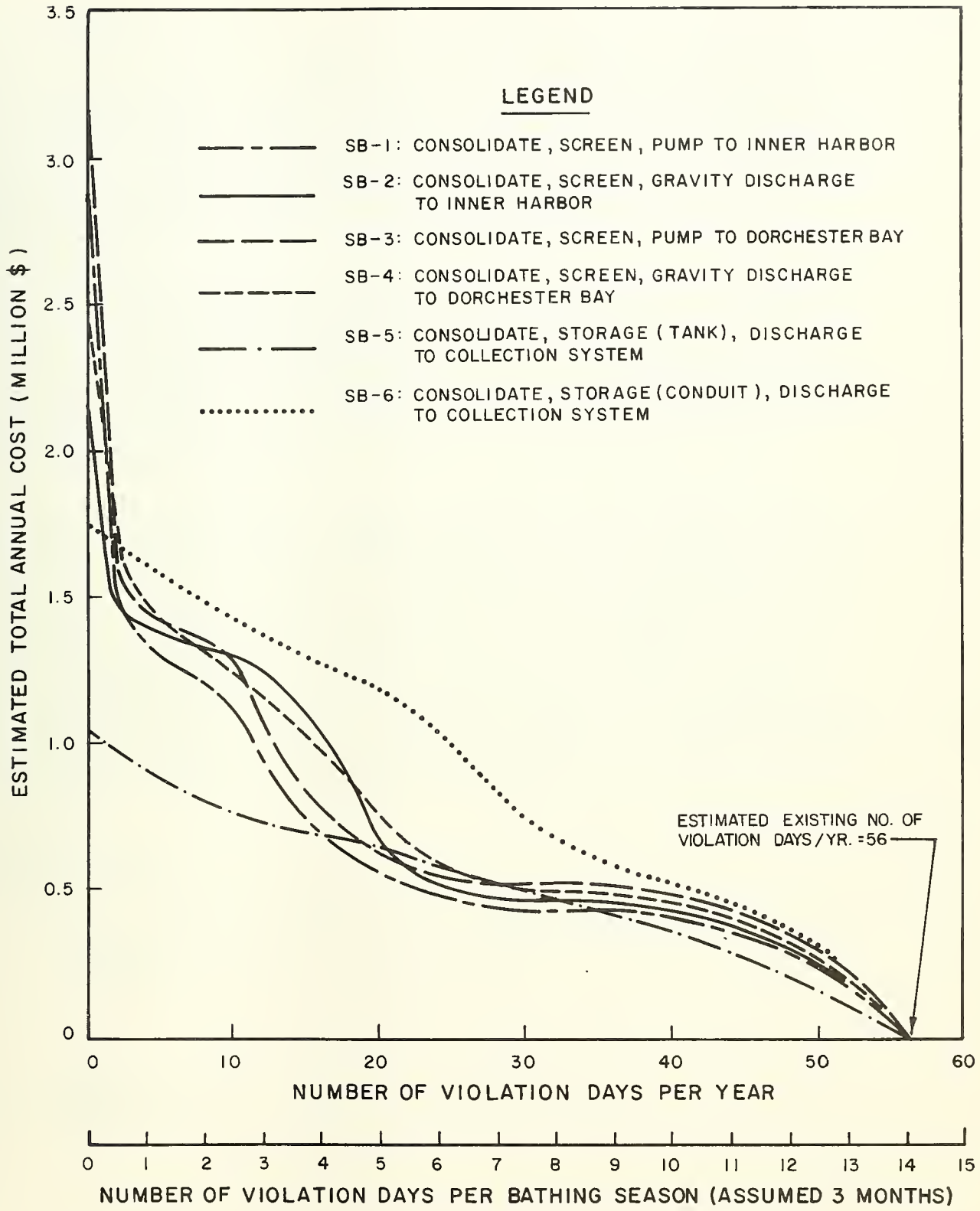


FIG. X-9 VIOLATION DAYS VS. COST FOR SOUTH BOSTON SCREENED ALTERNATIVES

X-30

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COMBINED SEWER OVERFLOW PROJECT
DORCHESTER BAY AREA FACILITIES PLAN

## Disinfection

Disinfection of CSO's is commonly practiced to control pathogens and other microorganisms in receiving waters. The disinfection agents used in wastewater and stormwater treatment include chlorine, hypochlorite (calcium and sodium), chlorine dioxide, and ozone. These four potential disinfection agents have some common characteristics; all are oxidizing agents, corrosive to equipment, and are highly toxic to both microorganisms and other life. Shown in Table X-7 are the characteristics of principal stormwater disinfection agents.

Disinfection (in conjunction with screening) can be considered as a unit process, capable of meeting the study area's CSO discharge limitations. The facilities would be equipped with screens for the removal of floatables; a disinfectant application system; and a detention tank sized to provide adequate contact time between the CSO's and the disinfectant. Stormwater disinfection facilities must be flexible and capable of automatic operation to handle intermittent and varying flows, pollutant characteristics and volumes.

Because of the aquatic environment of Dorchester Bay, the toxicity of possible disinfecting agents should be evaluated. For example, chlorine, in the free available form, reacts readily with nitrogenous organic materials to form chloramines. These compounds are considered toxic to fish. The toxicity to aquatic life of chlorine will depend upon the concentration of total residual chlorine, which is the amount of free chlorine plus chloramines and other chloro-organic compounds. The EPA has recommended ("Quality Criteria for Water", July 1976) a maximum chlorine residual of 0.01 mg/l in the water column for the protection of marine aquatic life. Other data sources(1) report that aquatic organisms may tolerate short term (i.e., 2 hours per day) exposure to higher levels of residual chlorine than the concentrations which have adverse chronic effects.

Selection of a CSO disinfection system should be based on the following considerations:

- CSO's are highly variable both in quantity and quality. Thus, any disinfection system must have the capability to meet these fluctuations.

(1) Brungs, W.A. 1979 "Effects of Residual Chlorine on Aquatic Life". JWPCF 45:2180

TABLE X-7

CHARACTERISTICS OF PRINCIPAL COMBINED SEWER OVERFLOW DISINFECTION AGENTS (1)

| Characteristic | Chlorine | Hypochlorite | Chlorine dioxide | Ozone | Bromine Chloride |
|---|---|---|---|---|---|
| Stability | Stable | 6 month half-life | Unstable | Unstable | Unstable |
| Reacts with ammonia to form chloramines | Yes | Yes | No | No | Yes |
| Destroys phenols | At high concentrations | At high concentrations | Yes | Yes | At high concentrations |
| Produces a residual | Yes | Yes | Short lived | No | Short lived |
| Affected by pH | More effective at pH 7.5 | More effective at pH 7.5 | Slightly | No | More effective between pH 6-10 |
| Hazards | Toxic | Slightly toxic | Toxic, explosive | Toxic | Toxic |

(1) From "Urban Stormwater Management and Technology Update and User's Guide", U.S. - EPA, September 1977. Table 121.

- In addition to the variability of the CSO's, during an overflow event the receiving waters are undergoing tidal exchanges. Accordingly, chlorine residual monitoring and control systems must be provided to maintain the EPA recommended chlorine residual concentration of 0.01 mg/l or less in the receiving waters.

- Chlorine, chlorine dioxide, and ozone are all dangerous gases that must be carefully handled by competent operators. Lesser hazards are associated with hypochlorite, which requires bulk storage and is somewhat unstable unless diluted.

- The coliform group of indicator organisms have a relatively low chlorine resistance when compared to such pathogens as enteric viruses and protozoan cysts. Thus, chlorine or other disinfectant dosages may be effective with coliforms but not with viruses and cysts. Total coliform levels may not be the most useful indication of disinfection requirements and efficiencies.

Based on the above considerations, it seems apparent that additional evaluations regarding the effectiveness and feasibility of disinfection are required. Additional analysis is required to estimate CSO dilution ratios in the bay, and required CSO disinfectant dosage rates. Accordingly, disinfection will be examined during the detailed evaluation phase.

Combined System Separation

Separation of the combined sewered portions of the Dorchester Bay study area was considered as a CSO abatement alternative. The purpose of separation is to provide conveyance of the sanitary flow directly to the Columbus Park Headworks and subsequently to the Deer Island treatment plant, and allow the stormwater to discharge directly to the receiving water without coming in contact with the sanitary sewage. With a separate sewer system, the current overflow of combined sanitary and stormwater flows would be eliminated, and the pollution of the Bay from this source would be prevented.

Theoretically, the quality of stormwater originating from rainfall should be higher than that of combined sewer overflows (in terms of the number of coliform present). In urban areas however, stormwater runoff often becomes contaminated by excrement from domestic animals and by illegal sanitary cross connections to the storm drain system. It is possible for stormwater runoff, during certain storm events, to contain coliform concentrations high enough to exceed SB standards during and immediately following a storm event at the location of discharge. Analysis of samples taken from separate storm drains during this study's wet weather sampling program have indicated that stormwater runoff from the Dorchester Bay study area contains a total coliform content of the same order of magnitude as that of combined sewer overflows ($2 \times 10^6$ MPN/100 ml) during some storm events.

Accordingly, separation of the combined sewered areas would most likely not significantly reduce the short-term impacts of total coliform, unless the stormwater could be discharged to the outer bay, away from the swimming and shellfish areas. Also, separation of combined sewer systems is a very difficult task in urban areas, and may produce adverse public response. The construction of the necessary additional sewers would obstruct local traffic for a considerable period. Plumbing separation in buildings would be even more difficult, and it often entails extensive renovations. For the City of Boston to assume the expense of the building's plumbing separation would be costly, and may raise legal problems as well. To require such separation at the owner's expense would obviously produce many complaints, and would be very difficult to enforce.

The cost of implementing the system separation alternative, including provision of separate sewers and the separation of building plumbing and individual connections, is estimated to be approximately $76,000 per acre, based on June, 1979 costs (ENR=2900). There are approximately 1,388 acres of combined sewered area within the Dorchester Bay study area as shown on Figures B, C, and D, appended. Thus, construction of a separated sewer system for this entire area alone would require an expenditure in the order of $140 million.* The ultimate cost would most likely be much greater than this considering the lengthy construction period that would be necessary. In addition, other existing system revisions to consolidate current separated areas with the new areas, and the construction of large diameter outfall conduits in the bay would be required.

The cost of separation was compared with the cost of the storage/containment alternative for the combined sewered area of 555 acres tributary to the CSO outlet BOS-090 location in the Dorchester portion of the study area. The cost of separation for this area would be approximately $55 million* based on the estimated unit-cost of $76,000 per acre. For this expenditure, the short term coliform impacts would most likely not be significantly reduced. The total estimated capital cost of providing screening and storage/containment facilities at outlet BOS-090, sized to allow zero violation days per year, would be approximately $10.3 million.* In addition, the estimated cost of separation is significantly greater than any of the screened structural alternatives at the zero violation day per year level.

Based on these considerations, it was concluded that combined system separation is not a viable or cost-effective CSO abatement alternative for the Dorchester Bay study area.

Beach Enclosures

An alternative that was considered in the 1972 report by Camp Dresser & McKee for the Massachusetts Division of Water Pollution Control entitled "Water Quality Improvement of Tenean and Malibu Beaches", calls for the protection of these two bathing areas from CSO's. The beaches are

*Cost includes an additional 30% to allow for engineering and contingencies.

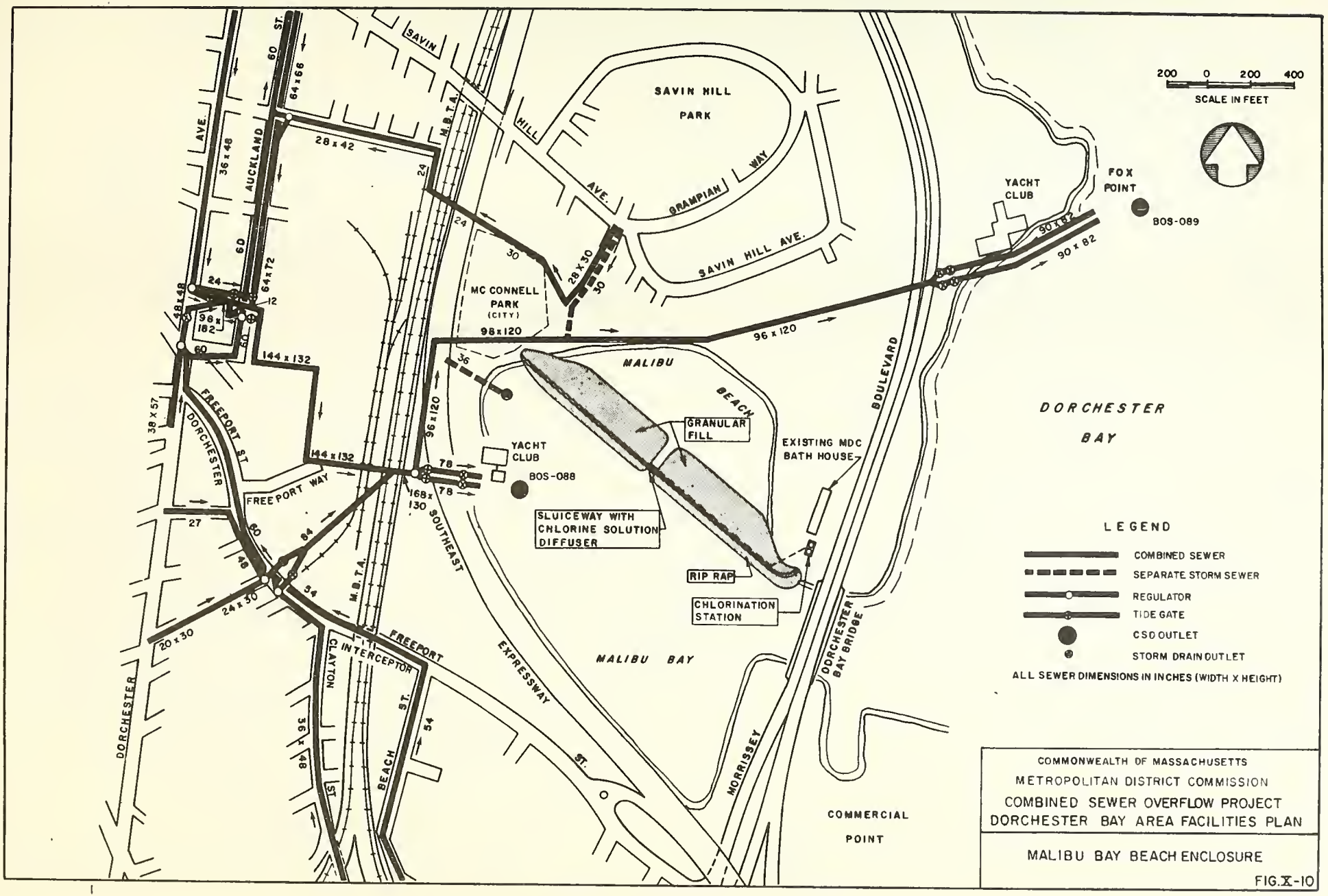
located in the Dorchester portion of the study area. Although this scheme would offer no protection to near-shore waters outside of the immediate beach areas, it is presented in this facilities plan for comparison with the screened structural unit process alternatives (storage-treatment). The selected beach enclosure alternative in the above-referenced report was "Dike Alternative 3", which involved the construction of an earthen dike of granular fill in Malibu and Tenean Bays. The top of the dikes would be 8 to 10 ft. wide, to provide for truck passage. At Tenean Bay, an existing wooden wall at a nearby marina is not capable of resisting the forces generated by a granular fill, thus about 600 ft. of the barrier would be designed as a steel sheet pile cellular cofferdam with a sand slope on the basin side.

Each enclosure would be provided with an inlet control structure containing sluiceways fitted with stop logs, a sluice gate for draining the basin completely when necessary, and diffusers for the distribution of chlorine solution for disinfection. Figures X-10 and X-11 display the beach enclosures at Malibu and Tenean beaches, respectively.

As shown on Table X-6, the estimated capital cost of the two beach enclosures is $13.8 million. This investment would offer complete protection of the beaches from external bacterial pollution and floatables, resulting in a 100 percent reduction in the number of violation days per year at the beaches. However, as mentioned above, there would be no reduction in the number of existing overflow days, or protection from floatables to other near-shore areas that are also affected by CSO discharges. The result would be the occurrence of the present estimated number of coliform violation days (i.e, 53 as shown on Figure X-4 for the Te-Se condition) at all near-shore areas other than the beaches. Thus, this alternative does not meet the Dorchester Bay study area objective of mitigating short-term coliform impacts and floatable reduction for <u>all</u> near-shore recreational areas. In addition, if screening facilities were provided on the existing CSO outlets, the estimated present worth cost ($18.5 million) would exceed that of Alternative D-2 for the zero violation day per year condition ($16.4 million). Since alternative D-2 would offer protection to all near-shore areas including beaches, and it has the potential of reducing the number of violation days per year to near zero, it can be concluded that the overall benefits realized by this storage alternative would greatly out-weigh those realized by the beach enclosures. Accordingly, the beach enclosure alternative is not considered a viable solution to meeting the objectives of the Dorchester portion of the study area, and was thus not selected for detailed evaluation.

<u>Best Management Practices</u>

<u>General</u>. Best Management Practices (BMP's) are considered as non-structural means of reducing the amount of pollution generated by non-point sources. By attacking the problem closer to its source, stormwater pollution can sometimes be controlled more cost-effectively than at downstream treatment facilities.

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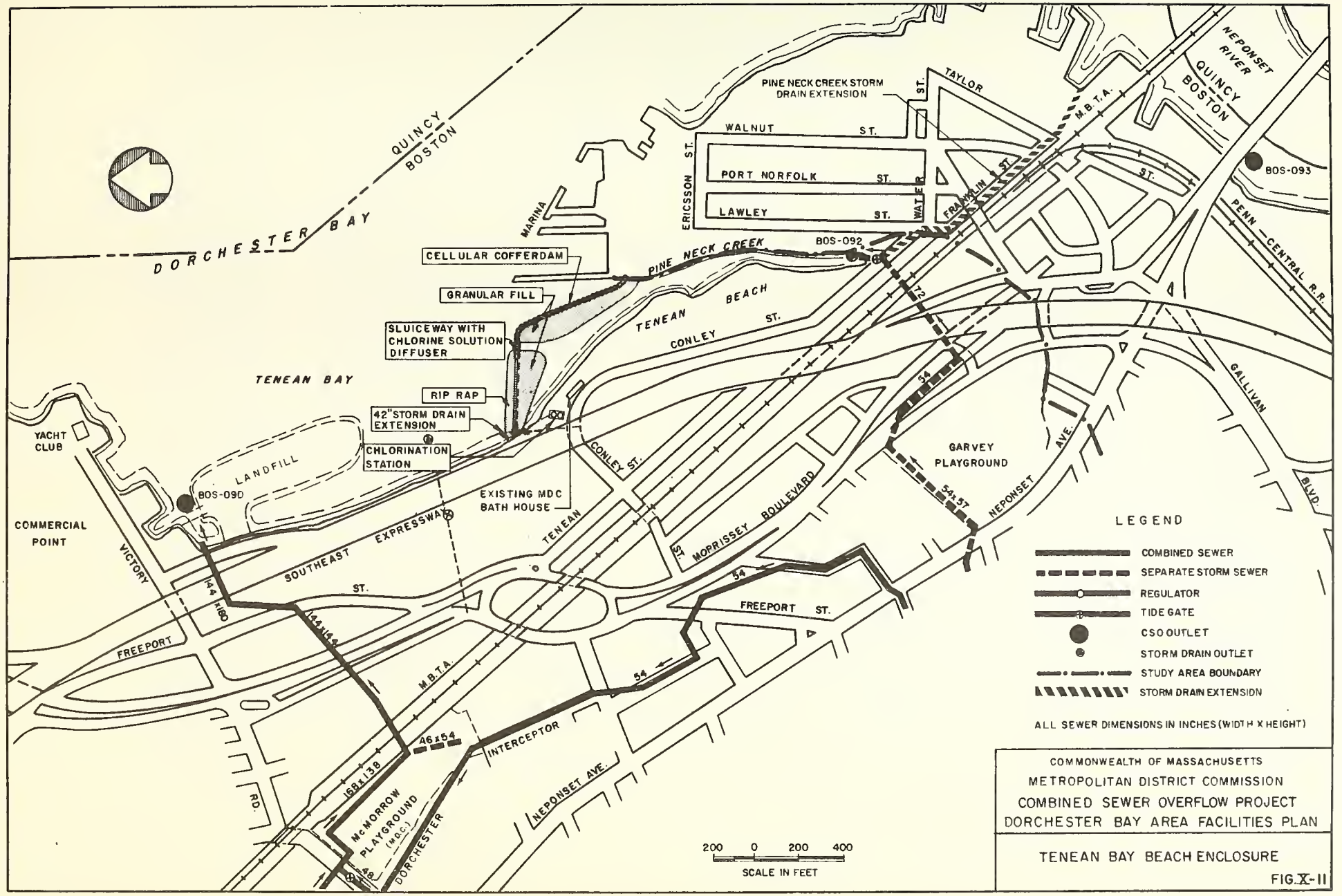
COMBINED SEWER OVERFLOW PROJECT
DORCHESTER BAY AREA FACILITIES PLAN

MALIBU BAY BEACH ENCLOSURE

FIG. X-10


DORCHESTER BAY
QUINCY
BOSTON
MARINA
CELLULAR COFFERDAM
GRANULAR FILL
SLUICEWAY WITH CHLORINE SOLUTION DIFFUSER
RIP RAP
42"STORM DRAIN EXTENSION
CHLORINATION STATION
EXISTING MDC BATH HOUSE
TENEAN BAY
YACHT CLUB
LANDFILL
BOS-090
COMMERCIAL POINT
VICTORY RD.
FREEPORT ST.
SOUTHEAST EXPRESSWAY
M.B.T.A.
INTERCEPTOR
McMORROW PLAYGROUND (MDC)
DORCHESTER
NEPONSET AVE.
PINE NECK CREEK STORM DRAIN EXTENSION
WALNUT ST.
PORT NORFOLK ST.
LAWLEY ST.
ERICSSON ST.
WATER ST.
TAYLOR ST.
FRANKLIN ST.
BOS-092
PINE NECK CREEK
TENEAN BEACH
CONLEY ST.
BOS-093
NEPONSET RIVER
PENN CENTRAL R.R.
GALLIVAN BLVD.
GARVEY PLAYGROUND
MORRISSEY BOULEVARD
LEGEND
COMBINED SEWER
SEPARATE STORM SEWER
REGULATOR
TIDE GATE
CSO OUTLET
STORM DRAIN OUTLET
STUDY AREA BOUNDARY
STORM DRAIN EXTENSION
ALL SEWER DIMENSIONS IN INCHES (WIDTH X HEIGHT)
200 0 200 400
SCALE IN FEET
COMMONWEALTH OF MASSACHUSETTS
METROPOLITAN DISTRICT COMMISSION
COMBINED SEWER OVERFLOW PROJECT
DORCHESTER BAY AREA FACILITIES PLAN
TENEAN BAY BEACH ENCLOSURE
FIG. X-11

Potential BMP's were listed previously in Table X-2. During Phase 1 of the screening process, BMP's that do not effectively meet the CSO abatement objectives, and/or are not applicable to the study area were discounted from further evaluation. Nine best management practices were selected for examination during the second phase of screening, as follows:

- Litter Control
- Street Sweeping
- Regulator Maintenance
- Catch Basin Cleaning and Maintenance
- Sewer Cleaning
- Sewer Flushing
- Tidegate Repair and Maintenance
- Existing System Real-Time Operation
- Legislation

Screening of Effective Best Management Practices. Each of the nine best management practices was examined to determine if it warranted further study during the detailed evaluation phase. A brief discussion regarding the purpose and potential effectiveness of the practices is given below.

Litter Control - Unless litter (i.e., cans, cigarettes, etc.) is prevented from reaching the street or is removed by street cleaning equipment, it often is found in stormwater discharges. Public education programs, the use of sidewalk waste disposal containers, and the enforcement of anti-litter laws are some source control measures. Litter control will also reduce the amount of screenings collected at the facilities associated with the structural alternatives (i.e, storage/containment).

Street Sweeping - Street sweeping removes accumulated dust, dirt, and litter from street surfaces, and it also improves the aesthetics of neighborhoods. The efficiency of the street sweeping operation depends generally upon the type of cleaning equipment used, the type of street pavement, the number of passes made, and the extent of vehicles parked on the streets.

The accumulation of deposits on the streets at the time of sweeping is dependent upon the interval of storm events between sweeping. For the Dorchester Bay study area, it is estimated that about 8 percent of the existing total suspended solids entering the bay annually from CSO's and stormwater can be removed by street sweeping the area's 240 curb miles once per week for a nine month period. The annual cost associated with such an operation is estimated to be $125,000.

Litter control and street cleaning can be considered together. The effectiveness of using these two best management techniques as a means of mitigating the impacts of CSO's is in doubt, because of the small reduction rates in floatables associated with the measures. Accordingly, litter control and street cleaning were not examined in

the detailed evaluation phase as a CSO abatement measure. However, these measures should continue to be implemented as methods of improving neighborhood aesthics and removing, although on a limited basis, street surface accumulations that are ultimately contained in CSO's.

Regulator Maintenance - Regulators are an integral part of the existing wastewater collection system. They serve as a means of diverting dry weather sanitary flows to the interceptor system, and during wet weather events when the capacity of the dry weather system is exceeded, the regulators divert combined sewer overflows to the bay. A discussion of the types of regulators found in the study area, along with the problems encountered with proper operation, was presented in Chapter IV. Most dry weather overflows witnessed during the field program were caused by regulators not operating properly, primarily because of lack of maintenance.

Because of the importance associated with the proper operation of regulators, this best management practice will be examined during the detailed evaluation phase.

Catch-Basin Cleaning and Maintenance - Historically, the role of catchbasins was to minimize sewer clogging by trapping coarse debris and to reduce odor emanations from low-velocity sewers. Catchbasins receive pollutants through the washoff of street surfaces and deliberate dumpings of oil wastes, leaves, grass clippings, etc. While no sampling of catch-basins was performed during this facilities plan, survey results from samplings of San Francisco's catch basins showed the following average pollutant characteristics: $BOD_5$ - 110 mg/l, total nitrogen - 8 mg/l, total phosphorous - 0.2 mg/l. Literature notes that a rainfall intensity of 0.01 to 0.02 in/hr lasting four hours is sufficient to displace 90 percent of the liquid contents of a catch-basin.

Because of the possibility of controlling nutrients (i.e., nitrogen and phosphorus) by proper catch-basin cleaning and maintenance, this best management practice warrants further examination during the detailed evaluation phase.

Sewer Cleaning - Sewer cleaning involves the routine inspection and cleaning of system conduits. The major causes of restrictions in large diameter sewers are siltation and accumulation of large debris. For example, the Dorchester Interceptor was cleaned within the past year, and approximately 2,020 cubic yards of debris was removed. By cleaning the interceptor, its hydraulic capacity increased some 20 percent, thus reducing the number of CSO events. Sewer cleaning is a required maintenance item that is important to ensure the proper operation of the wastewater collection system. This best management practice will be discussed subsequently in the detailed evaluation phase.

Sewer Flushing - Much attention has been given to sewer flushing recently. Flushing of sewers on a regular basis can ensure the continuing capability of sewer laterals and interceptors to carry up to their design capacity. It also alleviates solids build-up, thus reducing the amounts of solids in CSO's.

EPA Region I, through Anderson-Nichols, Engineers, engaged Clinton Bogert Associates to compare CSO pollution abatement techniques, including sewer flushing, with EMMA Alternative 1. ("Review of Alternatives for Evaluation of Sewer Flushing - Dorchester Area - Boston" - May 1979). The recommendations of this report call for additional studies to evaluate (1) the feasibility of flushing large diameter sewers; (2) automatic sewer flushing devices; (3) the use of existing in-line system storage; (4) the expanded use of system computer simulation (i.e., model STORM); (5) the extent of dry weather flow deposition; and (6) the practicality of using sewage for flushing.

Sewer flushing alone cannot eliminate the impacts associated with CSO's. Like other best management practices, sewer flushing techniques are an adjunct to structural alternatives.

The Clinton Bogert Associates report evaluated the effectiveness of sewer flushing in terms of TSS and $BOD_5$ removal; neither of which are CSO abatement objectives for the Dorchester Bay study area. Data correlating TSS or $BOD_5$ with total coliform is non-existent. However, assuming for the sake of gross comparison that there is such a relation, the effectiveness of sewer flushing could be compared with the effectiveness of the screened structural alternatives (i.e. storage) on a unit removal basis for the Dorchester portion of the study area.

Under structural Alternative D-2, the estimated present worth cost for a storage facility having 100 percent capture or zero overflow days per year is $16.4 million. On a long term basis, and utilizing the model SEMSTORM results for the Te-Se conditions, CSO's in the Dorchester portion of the study area presently produce approximately 700 pounds of TSS per day. Thus, 100 percent overflow removal can be obtained at a unit cost of $23,500 per pound of TSS. Clinton Bogert reported an average TSS removal rate of 6 percent, for a present worth cost of $3.0 million; resulting in a unit cost TSS removal rate of $71,400 per pound. Accordingly, even when constructing new storage facilities (vs. using existing in-line storage which will be discussed subsequently), sewer flushing does not seem to be a cost-effective measure for abating the impacts of CSO's in the study area, based on the results of Clinton Bogert's draft report.

Because of the unit pollutant removal costs associated with sewer flushing, and the fact that additional evaluations are required to determine its effectiveness (i.e., Clinton Bogert's recommendations) and the source and volume of flushing water, it has been determined the sewer flushing does not warrant further investigations in this facilities plan unless additional field prototype data becomes available that show this technique to be more attractive.

Tidegate Repair and Maintenance - When operating properly, tide gates minimize the intrusion of sea water into the wastewater collection and transport systems during tidal exchanges. Because of

the nature of a tidegate, which is essentially a flap gate, debris may collect or float into the gate, thus preventing full closure at low tide periods. When the tide level rises, sea water intrudes through the partially opened gate and into the collection system, thereby reducing the hydraulic capacity and available storage in the system. Proper tidegate repair and maintenance programs will be discussed later in the detailed evaluation phase.

Existing System Real-Time Operation (Existing In-line Storage). With proper operation of the existing system, unused volume in large sewerage system conduits can be utilized as storage facilities. This alternative includes the installation of effective regulators, level sensors, inflatable dams, tide gates, advanced storm warning systems, and computerized or automated system control. Such systems have been developed and successfully implemented in Seattle, Minneapolis, St. Paul, and Detroit.

The existing CSO conveyance conduits in the Dorchester portion of the study area have a volume equivalent to about 7 mil. gal. If these conduits were to be used as in-line storage, pumping would be required to empty the contents into the interceptor system when conveyance capacity becomes available. Questions regarding solids deposition and removal, and operational procedures should be addressed prior to determining the feasibility of this alternative. Thus, the use of existing in-line storage by real-time operation of the system should be explored during the detailed evaluation phase.

Legislation. Legislation is often required to implement best management practices. The responsibility for proper operation of a collection system and consumer disposal practices should be clearly stipulated by legislation. Thus, legislation relating to the implementation of recommended CSO abatement alternatives will be examined during the detailed evaluation phase.

## Alternatives Selected for Detailed Evaluation

Based on the results of the screening process, three CSO abatement alternatives were selected to proceed into the detailed evaluation phase of this facilities plan. The alternatives were selected based on their known or potential ability to meet the following criteria:

1. Conforms with the study area's CSO abatement objectives,
2. Cost effectiveness,
3. Less operational problems associated with the constructed facilities, and
4. Minimum environmental consequences.

Structural Alternatives. Two structural alternatives were selected: the unit processes of storage and disinfection with both to include floatables removal facilities (i.e., screens). A description of each is given below.

Storage

For the South Boston area, the effectiveness of a storage/containment facility located in the vicinity of Columbus Park would be evaluated, as described in screened alternative SB-5. The total estimated 1980 present worth cost associated with reducing the number of violation days per year to the 0-16 range will be evaluated.

For the Dorchester area, the effectiveness of two storage/containment facilities, as outlined in screened alternative D-2 would be evaluated. The total estimated 1980 present worth cost associated with reducing the number of violation days per year to the 0-16 range will be evaluated. As discussed subsequently, the concept of storage will also be evaluated.

As discussed subsequently, the concept of storage will also be evaluated under the best management practices of Existing In-line Storage: Real-Time System Operation. The results of this evaluation will be coordinated with the sizing of storage/containment facilities.

Disinfection

As discussed previously, additional investigations are required to estimate the effectiveness of disinfection and any resultant impacts on the bay's aquatic life, prior to determining the appropriateness of using the disinfection process. Further investigations will be performed during the detailed evaluation phase.

Best Management Practices. The action-impact relationship associated with BMP's are often difficult to quantify. BMP's are classified as nonstructural and low structurally intensive alternatives. Six were selected as being applicable to the Dorchester Bay study area:

- Regulator Maintenance
- Catchbasin Cleaning and Maintenance
- Sewer Cleaning
- Tidegate Repair and Maintenance
- Existing In-line Storage: Real-Time System Operation
- Legislation

Moon Island Joint Area Alternatives

General

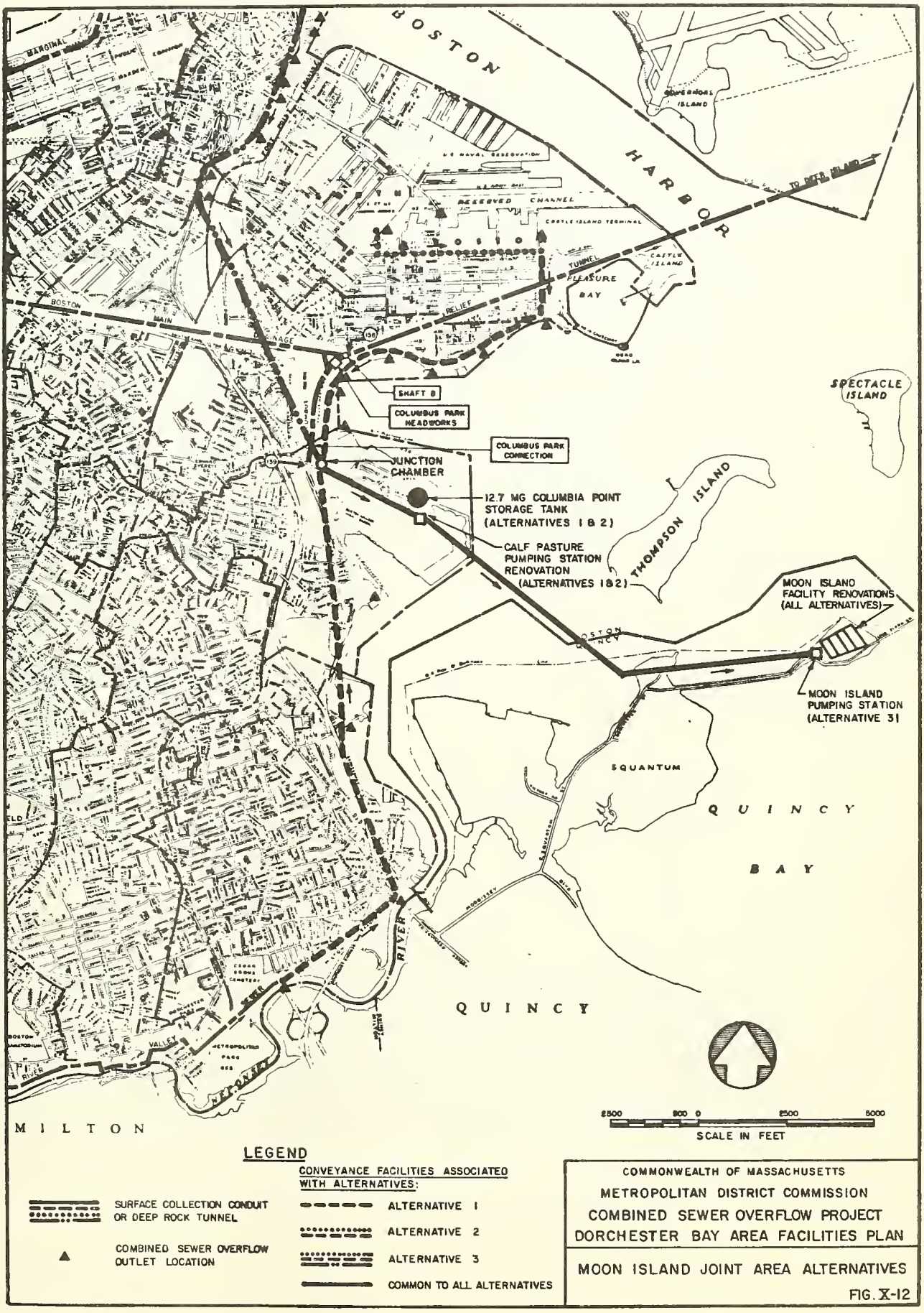
Alternatives involving the use of the existing Calf Pasture/Moon Island facilities for CSO abatement were investigated during the screening process on a joint area basis. Three service area alternatives were developed involving the Dorchester Bay, Neponset, and Inner Harbor study areas, and were generally based on the "Moon Island Tunnel Plan" and the "Modified Moon Island Tunnel Plan" as presented in the 1975 EMMA study. The intent of the current investigation is to compare the estimated costs and resulting benefits of the Moon Island schemes, to those of the screening alternatives that were selected for detailed evaluation from each of the project's study areas. Similar to the screening of study area alternatives, estimated costs for the Moon Island alternatives were based on conceptual layouts of conveyance and pumping facilities, and do not reflect the refinements of a more detailed evaluation. Results of this comparison will serve to determine whether any or all of the Moon Island alternatives should be considered for detailed evaluation.

Description of Alternatives

Three service area alternatives were developed, and facilities associated with each area were sized for exceedance conditions representing three levels of CSO capture: 15, 50, and 85 percent. Accordingly, 9 schemes were investigated -- three service area alternatives, each evaluated for three exceedance conditions. The service area alternatives are described as follows. The facilities associated with each are shown on Figure X-12.

Alternative 1. The facilities associated with this plan would be essentially the same as the "Modified Moon Island Tunnel Plan" presented in the EMMA study (except as described below). This alternative would serve the entire Dorchester Bay and Neponset study area, capturing flow from 12 CSO's and serve a total combined sewered area of about 1620 acres. As shown on Figure X-12, CSO's would be conveyed to the Calf Pasture Pumping Station by a series of surface collection conduits and deep rock tunnels. As proposed in EMMA, a 12.7 mil gal. detention tank would be located adjacent to the pumping station and serve to capture the "first flush" CSO volume. Stored flow from this tank would be discharged to the collection system when capacity becomes available. CSO volumes in excess of 12.7 mil. gal. would be pumped by the Calf Pasture pumping station and through a new Dorchester Bay tunnel to the Moon Island detention tanks. The Calf Pasture pumping station would be renovated to serve as a deep rock pumping station. The existing 7.5-ft diameter, brick lined Dorchester Bay tunnel would be abandoned and replaced. This differs from EMMA's recommendation of renovating the existing tunnel in that, based on its age (approximately 97 years), present condition (relative to structural integrity), and the potential amounts of sediment deposition , a new tunnel is probably justifiable. As described in Chapter IV, the carrying capacity of the tunnel as measured during the 1947 flow tests was about 154 mgd. The tunnel's measured capacity when new was about 200 mgd. Although the present carrying capacity is unknown, it can be assumed to be somewhat less than 154 mgd since it has never been cleaned. In addition, four of the nine alternative schemes evaluated for this analysis would require a carrying capacity in excess of 200 mgd, thus requiring either a new tunnel, or a new, parallel tunnel which would be used in conjunction with a rehabilitated existing tunnel. The question also arises as to whether the physical condition of the existing tunnel could withstand the unbalanced hydrostatic forces that may develop if the tunnel were to be dewatered for rehabilitation. Based on these reasons, it was assumed for these analyses', cost estimating purposes, that a new Dorchester Bay tunnel would be required for each scheme evaluated.

Flow from the Dorchester Bay tunnel would discharge to the Moon Island detention facility, which would be renovated to allow for solids settling and chlorination. The useable storage volume in the tanks would be in the order of 45 mil. gal. The settled, chlorinated effluent would be released to the harbor, on the outgoing tide.


BOSTON
HARBOR
SHAFT 8
COLUMBUS PARK HEADWORKS
COLUMBUS PARK CONNECTION
JUNCTION CHAMBER
12.7 MG COLUMBIA POINT STORAGE TANK (ALTERNATIVES 1 & 2)
CALF PASTURE PUMPING STATION RENOVATION (ALTERNATIVES 1 & 2)
THOMPSON ISLAND
SPECTACLE ISLAND
MOON ISLAND FACILITY RENOVATIONS (ALL ALTERNATIVES)
MOON ISLAND PUMPING STATION (ALTERNATIVE 3)
SQUANTUM
QUINCY BAY
QUINCY
MILTON
SCALE IN FEET
LEGEND
CONVEYANCE FACILITIES ASSOCIATED WITH ALTERNATIVES:
SURFACE COLLECTION CONDUIT OR DEEP ROCK TUNNEL
COMBINED SEWER OVERFLOW OUTLET LOCATION
ALTERNATIVE 1
ALTERNATIVE 2
ALTERNATIVE 3
COMMON TO ALL ALTERNATIVES
COMMONWEALTH OF MASSACHUSETTS
METROPOLITAN DISTRICT COMMISSION
COMBINED SEWER OVERFLOW PROJECT
DORCHESTER BAY AREA FACILITIES PLAN
MOON ISLAND JOINT AREA ALTERNATIVES
FIG. X-12

Alternative 2. This alternative contains all of the elements of Alternative 1, with the service area expanded by about 230 acres (total area of 1850 acres) to include three CSO's located in the northern portion of South Boston within the Inner Harbor study area. As shown on Figure X-12, an extension of the collection conduit in South Boston to intercept these three additional CSO's is the only difference between this scheme and the facilities described under Alternative 1.

Alternative 3. This alternative is generally based on the "Moon Island Tunnel Plan" as presented in EMMA. As indicated on Figure X-12, it is the largest service area alternative and includes the entire Neponset and Dorchester Bay study areas, and portions of the Inner Harbor Study area that are tributary to the northern portion of South Boston and Fort Point Channel. This scheme would intercept a total of 24 CSO's and serve a total combined sewered area of about 4300 acres. As shown on Figure X-12, all intercepted CSO's would be conveyed directly to Moon Island through a series of surface collection conduits and deep rock tunnels. A new pumping station, located at Moon Island, would pump from the tunnel system, into the detention facility which would be rehabilitated to allow for sedimentation and chlorination. This scheme differs from Alternatives 1 and 2 since the Calf Pasture pumping station would be abandoned, and there would be no storage facility at Columbia Point. Also, under Alternative 3, the stored flow at Moon Island would be pumped back to the collection system via a new conduit when capacity becomes available. CSO volume in excess of the total Moon Island storage capacity of 45 mil. gal. would discharge to the harbor after it had been settled and chlorinated.

Estimated Costs of Alternatives. The estimated present worth cost (ENR = 2900) of the alternatives for each exceedance (percent capture) condition is summarized as follows:

| Percent of Total CSO Captured | Estimated Present Worth Cost (million $) | | |
|---|---|---|---|
| | Alt. #1 | Alt. #2 | Alt. #3 |
| 15 | 30.0 | 32.0 | 68.0 |
| 50 | 45.0 | 70.5 | 95.1 |
| 85 | 82.2 | 89.2 | 150.0 |

It should be emphasized that these costs are based on a preliminary engineering investigation utilizing conceptual layouts of conveyance conduits, tunnels, pumping stations and other associated facilities. Accordingly, they do not reflect the refinements of a detailed evaluation which would be performed during the evaluation of selected alternatives phase (Chapter IX), if warranted, based on the results of this analysis. The criteria used to estimate design flowrates and and the required sizes of conveyance and pumping facilities is discussed in the following section.

Receiving Water Impact Determination

Receiving water impacts associated with each alternative were determined by the HMC for selected exceedance conditions. The land-based information was submitted to the HMC in the form of exceedance curves which represent the CSO discharge frequency and volume into each near-shore harbor model segment impacted. The statistical data associated with each curve was used to develop additional curves representing 15, 50, and 85 percent capture conditions at each near-shore segment. Referring to Chapter VIII, the near-shore harbor segments that correspond to each alternative are as follows:

| Harbor Segment | Alt. #1 | Alt. #2 | Alt. #3 |
|---|---|---|---|
| 11 | X | | |
| 11 | | X | X |
| 13 | X | X | X |
| 24 | X | X | X |
| 25 | X | X | |
| 26 | X | X | X |
| 27 | X | X | X |
| 28 | X | X | X |
| 207 | X | X | X |
| 210 | X | X | X |
| 213 | X | X | X |
| 214 | X | X | X |
| 4 | | | X |
| 10 | | | X |

Also submitted to the HMC were exceedance curves that represent the discharge from the Moon Island detention facility under each alternative and for certain exceedance conditions. These were developed by graphically compiling the exceedance curves for each near-shore harbor segment, in order to simulate the resulting CSO volume and frequency at Moon Island. This procedure was also used to estimate conveyance capacities and pumping rates for preliminary sizing and costing of facilities. The total resultant CSO volumes from the compilation procedure were converted to flowrates by assuming the volume would be generated over a "typical" storm duration of 7 hours(1). Using the volumes and this assumed duration, and assuming a triangular shape of the storm hydrograph, the peak flowrates were estimated.

Since Alternatives 1 and 2 include 12.7 mil. gallons of off-line storage located at Columbia Point that would be bled back to the collection system after the wet weather event, certain exceedance conditions resulted in no discharge from Moon Island. This is also true for Alternative 3 in which the 45 million gallons of storage at Moon Island would be bled back to the collection system. Accordingly, only the following exceedance conditions resulted in a discharge to the harbor from Moon Island:

(1) Table 1 of Hydroscience's November 15, 1978 "Boston Combined Sewer Overflow Study - Preliminary Impact Information"

| Alternative | Percent of Total CSO Captured |
|---|---|
| 1 | 85 |
| 2 | 50 |
| 2 | 85 |
| 3 | 85 |

It should be emphasized that since the development of the combined exceedance curves was based on the compilation of several near-shore harbor segment exceedance curves, there are obvious inherent inaccuracies when summing log-normal event distribution curves. However, for the purpose of screening, this degree of accuracy was deemed sufficient.

Table X-8 shows the results of the HMC analysis of the three Moon Island Joint Area Alternatives for impacted harbor model segments. The assessment was performed assuming total removal of dry weather overflows (DWO's). As shown for each alternative, the long-term coliform levels would be in compliance with the standards under all percent capture conditions at all locations except for Tenean Beach, Neponset River, and Fort Point Channel. It should be noted that Neponset River coliform levels cannot comply with the standards under any study area alternative, due to the high coliform levels contributed from tributary areas located upstream of the Neponset study area. Tenean Beach levels contravene the standards at the 15 percent capture condition, and are borderline at the 85 percent capture condition, for each alternative. Only the 85 percent capture condition of Alternative 3 results in compliance at Fort Point Channel. Accordingly, the Moon Island alternatives cannot be considered a "complete" solution to the CSO problem.

Comparison With Study Area Alternatives

The Moon Island alternatives were compared to the individual study area alternatives that were selected for detailed evaluation on the basis of costs and receiving water impacts. Estimated costs were compared on a present worth cost basis. Receiving water impacts were assessed both in terms of total annual coliform load into Boston Harbor, and total number of overflow days per year. In addition to the Dorchester Bay study area's selected alternatives (described earlier in this chapter), information was obtained from the Neponset study area and Inner Harbor study area consultants pertaining to their selected alternatives. Since the Moon Island Alternatives 2 and 3 would serve only a portion of the Inner Harbor study area, it was necessary to determine the costs and receiving water impacts of only those portions of the selected Inner Harbor area alternatives that would serve an area corresponding to the service areas of Moon Island Alternatives 2 and 3.

As described earlier in this chapter, the selected screened structural alternatives for the Dorchester Bay study area are "SB-5" for South Boston, and "D-2" for Dorchester, each consisting of a storage/containment scheme. Overflows would be collected, screened, stored during the wet weather period, then bled back into the interceptor system when

TABLE X-8 HARBOR MODEL CALCULATED TOTAL COLIFORM CONCENTRATIONS FOR MOON ISLAND JOINT AREA ALTERNATIVES[1]

| Location | Harbor Segment Number | Total Coliform Standard | Future No Action (Te,Se)[2] | Moon Island Joint Area Alternatives | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | #1 Percent Captured | | | #2 Percent Captured | | | #3 Percent Captured | | |
| | | | | 15% | 50% | 85% | 15% | 50% | 85% | 15% | 50% | 85% |
| **CALCULATED 50th PERCENTILE CONCENTRATIONS ($C_{50}$)[3]** | | | | | | | | | | | | |
| Fort Point | 10 | 10,000[4] | 6,500 | 6,500 | | 6,500 | | | 6,500 | 3,200 | | 1,100 |
| Pleasure Bay | 29 | 700 | <10 | <10 | * | <10 | * | * | <10 | <10 | * | <10 |
| Carson Beach | 24 | 700 | 70 | 70 | | 40 | | | 50 | 70 | | 60 |
| "L" St. Beach | 26 | 700 | 80 | 60 | | 30 | | | 30 | 60 | | 40 |
| Malibu Beach | 12 | 700 | 100 | 90 | | 50 | | | 60 | 90 | | 60 |
| Tenean Beach | 213 | 700 | 400 | 280 | | 170 | | | 200 | 280 | | 180 |
| Neponset River | 209 | 700 | 700 | 720 | | 370 | | | 400 | 720 | | 420 |
| **CALCULATED 80th PERCENTILE CONCENTRATIONS ($C_{80}$)[3]** | | | | | | | | | | | | |
| Fort Point | 10 | - | 41,800 | 41,800 | | 41,800 | | | 41,800 | 20,000 | | 6,900 |
| Pleasure Bay | 29 | 1,000 | <10 | <10 | * | <10 | * | * | <10 | <10 | * | < 10 |
| Carson Beach | 24 | 1,000 | 500 | 520 | | 350 | | | 400 | 520 | | 440 |
| "L" St. Beach | 26 | 1,000 | 500 | 360 | | 210 | | | 200 | 360 | | 240 |
| Malibu Beach | 12 | 1,000 | 800 | 530 | | 320 | | | 300 | 530 | | 360 |
| Tenean Beach | 213 | 1,000 | 2,200 | 1,600 | | 960 | | | 1,000 | 1,500 | | 1,000 |
| Neponset River | 209 | 1,000 | 3,700 | 3,500 | | 1,800 | | | 1,800 | 3,500 | | 2,000 |
| **CALCULATED 90th PERCENTILE CONCENTRATIONS ($C_{90}$)[3]** | | | | | | | | | | | | |
| Fort Point | 10 | 25,000[4] | 110,000 | 110,000 | | 110,000 | | | 110,000 | 51,000 | | 18,500 |

Notes

[1] From Tables 12-19 of Hydroscience's July 16, 1979 "Boston Combined Sewer Overflow Study - Harbor Model Results for Evaluation of Alternatives."

[2] With existing known project area (total) dry weather flows eliminated, total coliform calculated at the location during wet weather period.

[3] MPN per 100 ml.

[4] Based on fecal coliform standards and an assumed total:fecal ratio of 10:1.

*Analysis not performed by Hydroscience.

conveyance capacity becomes available. CSO volumes exceeding the storage capacity would overflow the tank(s) and discharge to Dorchester Bay. Tanks and other components were sized for a varying number of overflow events per year. Since the Neponset area and Inner Harbor area selected alternatives were preliminarily designed to capture the 1-year, 6-hour storm which, theoretically, would allow one overflow day per year to occur, costs for the Dorchester area alternatives were based on allowing one violation day per year to occur. These were estimated by interpolating between the costs associated with 0 and 6 violation days per year for Dorchester and between 0 and 10 violation days per year for South Boston (refer to Table X-6).

Dorchester Bay's total coliform load to the harbor was based on the estimated average CSO volume per overflow event determined by SEMSTORM model output, and applying the average total coliform concentration of 2.2 X $10^6$ MPN per 100 ml as determined from the field sampling program (Chapter VI).

The Neponset study area consultant has recommended their "Alternative 3" for Port Norfolk and "Alternative 5" for the Granite Avenue subarea be used in combination with the other study area's selected alternative(s) for comparison with the Moon Island Joint Area Alternatives. While these have the highest capital cost of all Neponset selected alternatives, they also provide a level of CSO control comparable to the other study area alternatives. A new separate sewer system in the Port Norfolk area with a 1-mgd sanitary flow pumping station and force main would eliminate all CSOs in that area (only storm water overflow would occur). The selected alternative for the Granite Avenue area would involve system improvements, including a new storm drain interceptor, and enough storage to capture the 1-year, 6-hour storm (i.e., one overflow day/yr). Overflow screening and disinfection would also be provided. Since the Granite Avenue tributary area is relatively small (175 acres), the number of coliform contributed by one overflow per year after disinfection is negligible compared to the contributions from the Dorchester Bay and Inner Harbor study areas.

The service area of the Inner Harbor consultant's Alternative 1 is nearly equivalent to the portion of the Inner Harbor study area served by Moon Island Alternative 3. Alternative 1 involves the consolidation and treatment of the following overflows, located along Fort Point Channel and the Reserved Channel: Nos. BOS/070, 069, 068, 065, 064, 063, 062, 061, 072, 073, 076, 078, and 079. Moon Island Alternative 3 does not include overflows BOS/072 and BOS/073 (total drainage area of 76 acres) but does include two additional overflows: BOS/060 and BOS/061 (total drainage area of 117 acres). Thus, this small difference in service areas should not significantly affect the comparison. A CSO treatment facility would be located at the southern end of Fort Point Channel, and would provide course screening, pumping facilities, solid removal and disinfection. All facilities were sized to capture and treat the 1-year, 6-hour storm.

Results of Comparison. Table X-9 compares the benefits and costs of Moon Island Alternative 1 at the 85 percent capture level with those of Dorchester Bay and Neponset study area alternatives based on capturing the 1-year, 6-hour storm. The average number of coliform discharged per year under Moon Island Alternative 1 would be in the order of 100 times greater than the individual study area alternatives, since facilities designed for the 85 percent capture level would allow approximately 9-12 overflows per year. Since overflows from the Moon Island Facility would be chlorinated (i.e., 99+ percent coliform kill), the coliform load to the harbor would be contributed from only the 9-12 overflows per year discharging from the study area's CSO outlets. The cost of Moon Island Alternative 1 would be almost $50 million greater than the total cost of the study area alternatives.

Moon Island Alternative 2 could not be directly compared to a combination of individual study area alternatives since the Inner Harbor area consultant did not isolate the costs and benefits for controlling only the CSO's generated from the three Inner Harbor CSO's. Thus, the total cost and benefits for individual study area alternatives comparable to Alternative 2 could not be reasonably determined. However, based on the results of the comparison with Alternative 1, and since Alternative 2 involves a relatively small increase in service area from that of Alternative 1, it can be assumed that the costs of Alternative 2 would most likely be considerably higher than the total cost of comparable study area alternatives. The estimated present worth cost of Moon Island Alternative 2 is $88.9 million, at the 85 percent capture level.

Table X-10 presents a comparison of Moon Island Alternative 3 with the Dorchester Bay, Neponset and equivalent Inner Harbor study area alternatives. Implementation of Moon Island at the 85 percent capture level would result in an estimated total annual coliform load that is two orders of magnitude greater than the study area alternatives, at a cost that is approximately 50 percent greater than the total cost of the study area alternatives.

In order to provide a level of CSO abatement that is comparable to the ability to capture the 1-year, 6-hour storm (i.e., one overflow per year), the Moon Island alternative facilities would have to be designed to capture approximately 99 percent of the CSO's. By utilizing the costs necessary to provide 15, 50 and 85 percent capture, the cost of a Moon Island alternative for 99 percent CSO capture is estimated to be in the order of $200 million.

## Conclusions

Based on the results of this comparison, it is evident that the individual study area alternatives selected for detailed evaluation are a more cost-effective and environmentally sound solution to the CSO problem than the Moon Island Alternatives. Accordingly, the Moon Island Joint Area Alternatives were not considered for detailed evaluation.

TABLE X-9 COMPARISON OF MOON ISLAND ALTERNATIVE 1
WITH INDIVIDUAL STUDY AREA ALTERNATIVES

| ALTERNATIVES | BENEFITS | | COSTS |
|---|---|---|---|
| | Total Coliform Loading to Boston Harbor(1) | No. of Overflows Per Year | Present Worth (Million $)(2) |
| Dorchester Bay Study Area Alternatives SB-5 and D-2 | .47 | $\leq$ 1 | 26.2 |
| Neponset Study Area Alternatives: | | | |
| Port Norfolk Alt. 3 | $\ll$ .01 | $\leq$ 1 | 7.0 |
| Granite Ave. Alt. 5 | | | |
| Total | .47 | | 33.2 |
| Moon Island Alternative 1 (85% Capture) | 50.00 | 9-12 | 81.8 |

Notes:

(1) x $10^{15}$ MPN per 100 ml/year

(2) ENR = 2900

TABLE X-10 COMPARISON OF MOON ISLAND ALTERNATIVE 3 WITH INDIVIDUAL STUDY AREA ALTERNATIVES

| ALTERNATIVES | BENEFITS | | COSTS |
|---|---|---|---|
| | Total Coliform Loading to Boston Harbor(1) | No. of Overflows Per Year | Present Worth (Million $)(2) |
| Inner Harbor Study Area Alternative 1 | .88 | $\leq 1$ | 62.0 |
| Dorchester Bay Study Area Alternatives SB-5 and D-2 | .47 | $\leq 1$ | 26.2 |
| Neponset Study Area Alternatives: Port Norfolk Alt. 3 Granite Ave. Alt. 3 | $\ll .01$ | $\leq 1$ | 7.0 |
| Total | 1.35 | | 95.2 |
| Moon Island Alternative 3 85% Capture | 364.00 | 9-12 | 150.4 |
| 99% Capture | 4.50 | $\leq 1$ | 200.0 |

Notes:

(1) $\times 10^{15}$ MPN per 100 ml/year

(2) ENR = 2900

## Existing Deer Island Tunnel System Maximization Alternative

### General

The existing inerceptor tunnel system, which connects the three headworks (Chelsea Creek, Ward Street and Columbus Park) in the CSO project area with Deer Island, transports dry weather flow to the treatment plant. If the tunnels could be effectively utilized for the additional transport of storm-related flow, then the frequency of overflows in the four planning areas could be reduced. The storm-related flows could be detained ahead of the treatment plant or treated directly at it. Utilization of the existing tunnel system to its maximum capacity was, therefore, investigated as a joint area alternative since it spans all four study areas.

The existing tunnels and headworks are shown on Figure X-13. Two methods were considered for delivering more wet weather flow to the tunnels:

(1) Increase pumping capacity through the tunnels; and

(2) Increase the capacity of interceptors which enter the headworks and increase the pumping capacity through the tunnels.

### Pumping Capacity Increase

The present pumping arrangement at Deer Island has a capacity of 900-mgd, if under design capacity all ten pump spaces are utilized by 90-mgd pumps. This 900-mgd pumping capacity is fixed by the headloss through the tunnels. However, additional flow through the tunnels could be delivered by pumping into the tunnels at the sites of the existing headworks.

In order to determine a maximum capacity for these hypothetical pumping stations, interceptor modeling was performed on the interceptors which lead into the Chelsea Creek and Columbus Park headworks. (The computer model of the interceptor system does not extend upstream of the Ward Street headworks, however, the results obtained at the Chelsea Creek and Columbus Park headworks were taken to be indicative of possible conditions at the Ward Street headworks.) The 1-year, 6-hour storm was selected as the design condition for comparison to other individual planning area alternatives being developed by the other area consultants.

Modeling results demonstrated that if the headworks were to be upgraded with additional capacity, and were to be equipped with pumping equipment to convey flows which result from the 1-year 6-hour storm, then the following pumping capacities would be required:

| Headworks | Existing Design Capacity | Estimated Pumping Capacity Required to Convey 1-year 6-hour Storm Flows to Deer Island |
|---|---|---|
| Chelsea Creek | 350 mgd | 660 mgd |
| Columbus Park | 182 mgd | 340 mgd |
| Ward Street | 256 mgd | 500 mgd |
| TOTALS | 798 mgd | 1,500 mgd |

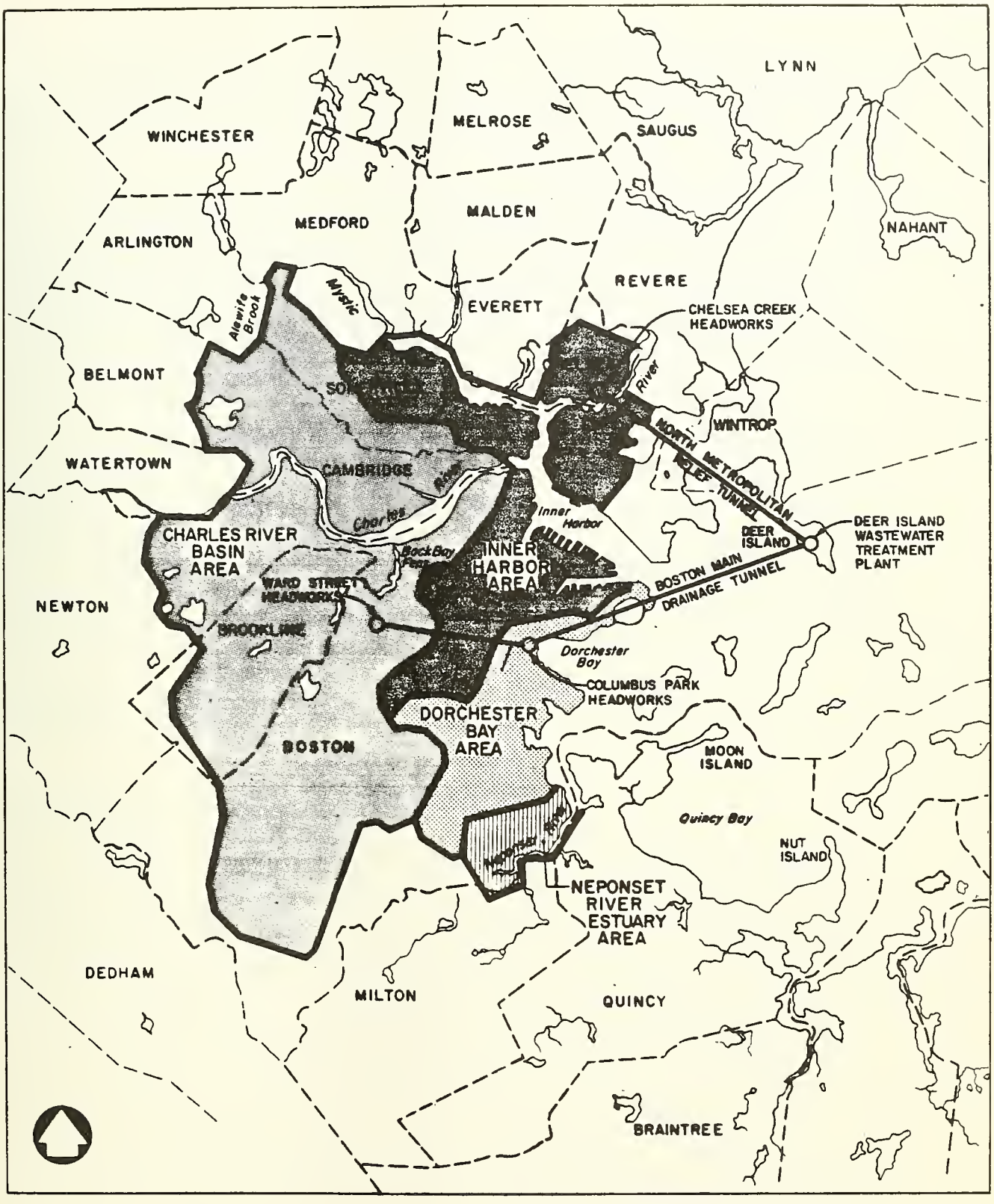



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FIG. X-13 EXISTING HEADWORKS AND TUNNEL SYSTEM

These estimated capacities assume that there would be no increase in interceptor capacity upstream of the headworks. Rather, by making more capacity available at the headworks and pumping into the tunnels, flow could be drawn from the existing upstream interceptors at a greater rate than is now possible.

The modeling results also demonstrated that by increasing headworks capacity, additional flow would only be drawn from those combined sewers closest to the headworks, such as in the Boston Main interceptor from the Dorchester Brook Sewer to the Columbus Park headworks. At points in the interceptors system distant from the headworks, such as in the Dorchester Interceptor, there would be no significant increase in flow. Accordingly, the benefit of this increase in capacity would be very limited, and essentially no improvement in the existing situation near Dorchester Bay beaches would be achieved.

Based on preliminary cost estimates for deep rock pumping stations developed for the Moon Island joint area alternatives, it is estimated that the cost for three new deep rock pumping stations at the sites of the existing headworks would be at least $133,000,000, (June 1979 price levels - ENR 2900) including headworks expansion (additional grit channels and screens). Not included is the dismantling of the present Deer Island pumping equipment or modifications required thereto. Also included in the cost is the present worth of annual costs for pumping and disinfection of the additional wet weather flow which would be transported. If primary treatment were provided, then annual costs would be even greater.

Based on its high cost and limited benefit, this alternative was not considered further during detailed evaluation.

Interceptor Capacity Increase

The result of the previous analysis showed that an increase in the capacity of the existing interceptors upstream of the headworks would be needed to achieve significant additional capacity to transport wet weather flow. Wet weather relief interceptors could be constructed parallel to the existing interceptors in order to provide this additional capacity. A preliminary analysis was made of the cost to construct relief interceptors throughout the four planning areas. It was assumed that the relief interceptors would be the same size as the existing interceptors so that the capacity of the system could be doubled. For preliminary cost estimating purposes, it was further assumed that the largest new relief interceptors would not be greater than 11' in diameter, that new pipes under 10' in diameter would be placed by open-cut excavation, and pipes 10' to 11' would be 50 percent tunneled and 50 percent open-cut. Unit costs for new sewers in the Dorchester Bay area were used in the cost estimate. It was estimated that approximately 60 to 80 miles of new interceptors would be required, which would cost at least $250 to $350 million (June 1979 price levels - ENR 2900), not including the cost of additional pumping capacity at existing pumping stations, nor including the cost of headworks expansion.

The benefit of such a scheme was examined for the Dorchester Bay area. If the interceptor capacity were doubled in South Boston and Dorchester, then the number of overflows per year in these areas would be cut by 30 to 40 percent, which results in about 30 overflows per year. Other less costly alternatives for the Dorchester Bay area would result in 0 to 16 overflows per year as shown earlier in this chapter.

Based on its high cost and limited benefit, this alternative was also rejected from further detailed evaluation.

The results of this joint area alternative feasibility study are consistent with the results of the Deep Tunnel Plan (refer to the Inner Harbor Study Area Facilities Plan) and Moon Island alternatives studies. All three of these joint area alternatives provide capacity to convey and store large volumes of wet weather flow. All three have very high capital costs for transporting large volumes of wet weather flow. It is the high cost of providing the transport capacity for pollution control during storms of up to 1-year frequency that make these joint areas alternatives unfavorable compared to the selected individual alternatives which involve decentralized facilities.

# CHAPTER XI - EVALUATION OF SELECTED ALTERNATIVES

## General

The purpose of this chapter is to present the results of the detailed evaluation of those CSO abatement alternatives that were selected during the screening phase. Based on the results of the screening process, three CSO abatement alternatives were selected for detailed evaluation as follows:

- o storage/containment
- o screening/disinfection
- o best management practices

During the detailed evaluation phase, several storage/containment and screening/disinfection alternatives were developed and evaluated on the basis of estimate capital and annual costs, operational considerations, legal and institutional considerations, environmental consequences and receiving water impacts. Also evaluated in more detail than in the screening phase were:

- o the requirements to implement a dry weather overflow abatement program
- o modifications to the Hoyt Street regulator located on the Dorchester interceptor
- o relocation of the Pine Neck Creek storm drain

At the completion of this phase, a recommended program was selected. Environmental issues associated with the alternatives evaluated during this phase of the project are discussed in Volume II, Chapters IV and V.

## Development of Selected Alternatives

### General

Alternatives evaluated consist of both structural and non-structural, summarized as follows:

Structural Alternatives
- storage/containment
- screening/disinfection
- Hoyt St. regulator modifications
- Pine Neck Creek storm drain relocation

Non-structural alternatives - best management practices

- dry weather overflow abatement program

The major structural alternatives of storage/containment and screening/disinfection were developed and evaluated based on estimated costs associated with reducing the annual number of CSO violation days to within the 0 to 12 range. Alternatives were developed for each of the following three areas:

1. South Boston - area tributary to CSO outlets BOS-081 through BOS-087

2. Fox Point - area tributary to CSO outlets BOS-088/089 in Dorchester

3. Commercial Point - area tributary to CSO outlet BOS-090 in Dorchester.

The benefits realized by modifying the Hoyt Street regulator and relocating the Pine Neck Creek storm drain were discussed in Chapter X, and it was concluded that both items should be included as part of the recommended plan. The Hoyt Street regulator modifications would result in about a 55 percent reduction in the estimated annual number of CSO violation days in the Dorchester portion of the study area (i.e., reduced from 53 to 24), and is a relatively simple and inexpensive way to significantly reduce wet weather - related receiving water impacts. Relocating the Pine Neck Creek storm drain in considered necessary since it discharges adjacent to Tenean Beach and is a source of runoff-related bacterial contamination as well as other pollutants contained in the runoff from its highly urbanized drainage area. A further discussion of these items, including a detailed description of required facilities and estimated costs, is presented later in this chapter, along with the non-structural alternatives which apply to the entire study area.

## Structural Alternatives - Storage/Containment

As shown on Figure XI-1, the Dorchester Bay study area has several areas that are served by separate storm and sanitary sewers. Most of these areas however, are tributary to CSO conduits which results in separate storm runoff being mixed with combined sewer flows before discharging to Dorchester Bay. During the screening phase, facilities associated with each screened alternative were sized for overflows from areas served by combined sewers only. This methodology was considered appropriate during the screening phase, since costs of screened alternatives were compared on a relative basis, and the level of screening evaluation did not dictate further study. However, during the detailed evaluation phase, faci-

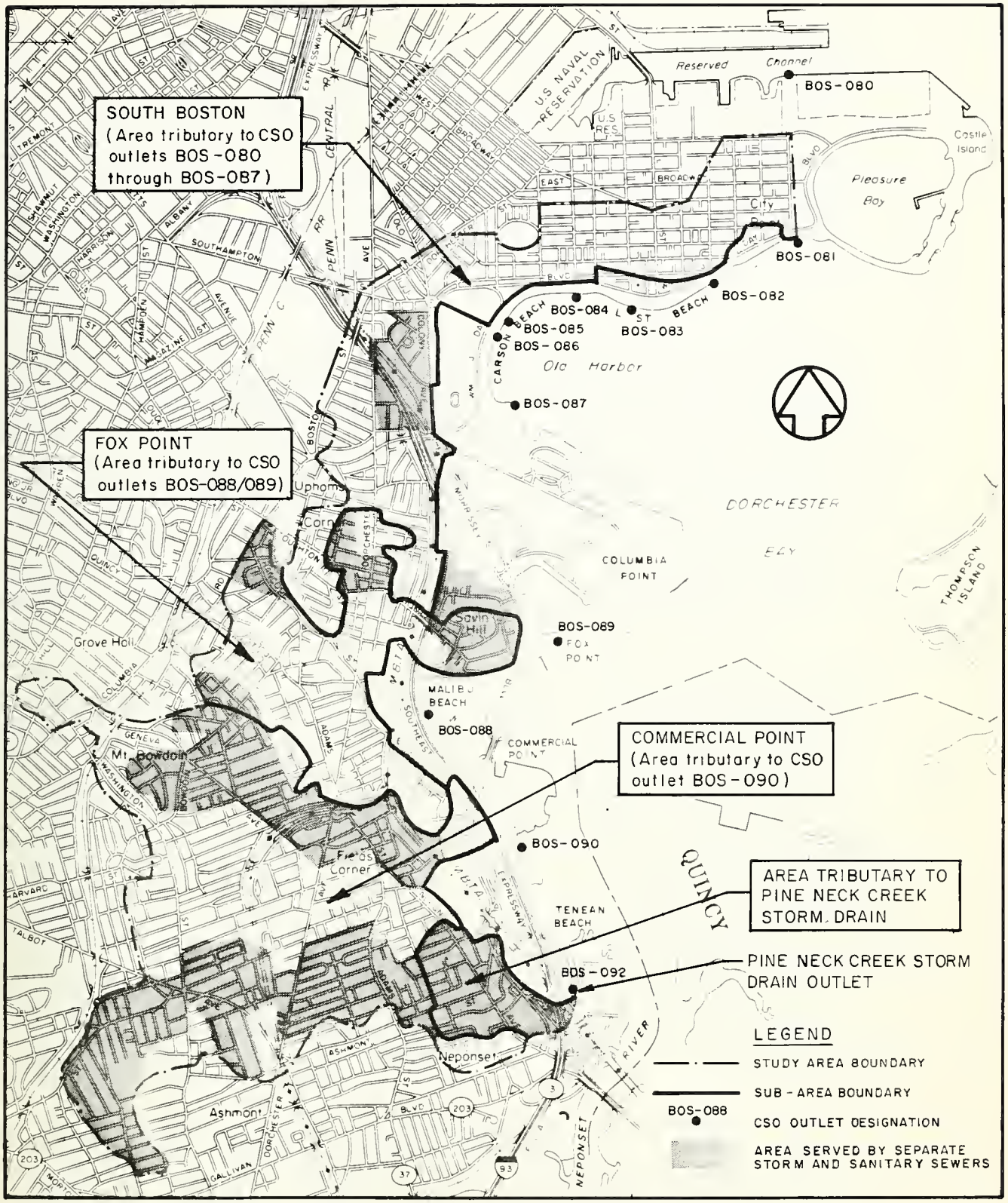


FIG. XI- 1 DISAGGREGATION OF STUDY AREA FOR DETAILED EVALUATION OF ALTERNATIVES

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lities and associated costs required to handle flows from both separated and combined areas were determined. This resulted in the development of several alternatives for each of the three major tributary (Figure XI-1) areas, basically involving either storing the entire flow from both separate and combined areas, or routing the flow from separate areas directly to the bay and providing storage facilities for the overflows from combined areas only. The feasibility and cost-effectiveness of utilizing available in-system storage in the Dorchester portion of the study area (i.e., Fox Point and Commercial Point tributary areas) was also investigated. A summary of the alternatives evaluated during this phase of the facilities plan is shown on Table XI-1.

South Boston. Two storage/containment alternatives (SB-A and SB-B) were developed and evaluated for the South Boston portion of the study area. Both alternatives involve consolidation of CSO outlets BOS-081 through BOS-087, the screening and storage of wet weather flows, and discharge of the storage volume to the existing collection system. SB-A involves the storage of CSO's only, and routing the runoff from separate sewered areas directly to the bay (total service area of 484 acres). SB-B provides for storage of both CSO's and separate stormwater runoff (total service area of 543 acres). Treatment of CSO's generated from the area tributary to BOS-080, which discharges into the Reserved Channel (tributary Area=47 acres), was not considered during this phase, however further analysis will be performed during the preliminary design phase (Chapter XII).

The South Boston CSO facilities were sized and costs estimated for the control levels of 1, 4 and 12 violation days per year. As discussed in Chapter X, required storage volumes for each control level were based on the storage-treatment curves generated utilizing the model SEMSTORM. These curves were refined from those shown on Figure X-5, to provide for increased accuracy within the 1 to 12 violation days per year range. A second curve was also developed to reflect the additional storage volume required to handle the runoff from the separate sewered areas for each control level (Alternative SB-B). The size of the consolidation conduit, was based on the runoff generated by storms that occurred within the 29-year period of rainfall records for the control level investigated. The storms were selected by running SEMSTORM for the South Boston area for each control level, and utilizing in the model, the storage volumes associated with each level. Storms that produced CSO quantities close to the storage volume provided were then selected as consolidation conduit design storms. When more than one storm satisfied this requirement, the storm that would result in the highest peak flow to be intercepted by the conduit was used for sizing.

The facilities associated with each alternative are similar. However, since SB-A involves routing the runoff generated from 59 acres of separate sewered areas directly to the bay through new storm drains, the volume of the storage facility will be smaller under this alternative. The required storage volumes used for detailed evaluation and for each control level and alternative are summarized as follows:

TABLE XI-1

EVALUATION OF SELECTED ALTERNATIVES

| Selected Alternatives | | Area Served, Acres (% of Total) | | | CSO Violation Days Per Yr | Estimated Annual Total Coliform Load to Bay From Tributary Area (Pseudo-Lbs x $10^8$) | Estimated Costs (Million $) | | |
|---|---|---|---|---|---|---|---|---|---|
| Designation | Description | Combined | Separated | Total | | | Capital(1) | Annual O&M | Present Worth(2) |
| SOUTH BOSTON | | | | | | | | | |
| SB-A | Consolidate and store CSO's and discharge to collection system, route storm drainage to bay | 484(100%) | 0(0%) | 484 | 1 | 8.4 | 28.54 | 0.10 | 22.87 |
| | | | | | 4 | 20.4 | 23.80 | 0.08 | 19.17 |
| | | | | | 12 | 67.6 | 18.00 | 0.06 | 14.30 |
| SB-B | Consolidate and store CSO's plus separate storm runoff, discharge to collection system | 484(89%) | 59(11%) | 543 | 1 | 1.1 | 29.35 | 0.12 | 23.75 |
| | | | | | 4 | 13.9 | 21.59 | 0.09 | 17.80 |
| | | | | | 12 | 63.3 | 17.53 | 0.07 | 14.05 |
| SB-C | Consolidate, screen and disinfect CSO's and storm drainage, discharge to Reserved Channel | 484(89%) | 59(11%) | 543 | 1 | | | | |
| | | | | | 4 | 2.3 | 29.50 | 0.13 | 24.71 |
| | | | | | 12 | 15.1 | 22.27 | 0.11 | 18.67 |
| | | | | | | 64.5 | 15.59 | 0.09 | 12.93 |
| DORCHESTER | | | | | | | | | |
| Area Tributary to CSO Outlet BOS-088/089 (Fox Point) | | | | | | | | | |
| D-A | Route Savin Hill storm drainage to bay, store CSO's and remaining storm drainage utilizing off-line storage and discharge to collection system | 326(85%) | 59(15%) | 385 | 1 | 3.8 | 14.01 | 0.07 | 11.61 |
| | | | | | 4 | 14.1 | 9.96 | 0.05 | 8.32 |
| | | | | | 12 | 43.9 | 7.46 | 0.04 | 6.29 |
| D-B | Same as D-A but utilize available in-system storage in conjunction with off-line storage and discharge to collection system | 326(85%) | 59(15%) | 385 | 1 | 3.8 | 15.96 | 0.08 | 13.20 |
| | | | | | 4 | 14.1 | 11.48 | 0.06 | 9.58 |
| | | | | | 12 | 43.9 | 6.15 | 0.06 | 5.32 |
| D-C | Route Savin Hill storm drainage to bay, store CSO's and remaining storm drainage utilizing available in-system storage and discharge to collection system | 326(85%) | 59(15%) | 385 | ≥ 9 | 31.8 | 6.15 | 0.06 | 5.32 |
| D-D | Screen and disinfect CSO's and separate storm drainage and discharge to bay | 326(85%) | 59(15%) | 385 | < 1 | 0.7 | 4.22 | 0.08 | 3.88 |
| Area Tributary to CSO Outlet BOS-090 (Commercial Point) | | | | | | | | | |
| D-E | Store CSO's and storm drainage utilizing off-line storage and discharge to collection system | 541(61%) | 340(39%) | 881 | 1 | 3.3 | 25.57 | 0.11 | 20.99 |
| | | | | | 4 | 21.2 | 14.81 | 0.07 | 12.31 |
| | | | | | 12 | 78.1 | 9.59 | 0.05 | 8.09 |
| D-F | Same as D-E but utilize available in-system storage in conjunction with off-line storage and discharge to collection system | 541(61%) | 340(39%) | 881 | 1 | 3.3 | 25.18 | 0.12 | 20.82 |
| | | | | | 4 | 21.2 | 14.43 | 0.08 | 12.14 |
| | | | | | 12 | 78.1 | 9.10 | 0.06 | 7.83 |
| D-G | Route storm drainage directly to bay, store CSO's utilizing off-line storage and discharge to collection system | 541(100%) | 0(0%) | 541 | 1 | 23.8 | 21.92 | 0.07 | 17.63 |
| | | | | | 4 | 43.1 | 16.70 | 0.05 | 13.39 |
| | | | | | 12 | 101.3 | 13.22 | 0.04 | 10.59 |
| D-H | Same as D-G but utilize available in-system storage in conjunction with off-line storage and discharge to collection system | 541(100%) | 0(0%) | 541 | 1 | 23.8 | 21.54 | 0.08 | 17.43 |
| | | | | | 4 | 43.1 | 16.21 | 0.06 | 13.12 |
| | | | | | 12 | 101.3 | 12.10 | 0.06 | 9.80 |
| D-I | Store CSO's and storm drainage utilizing available in-system storage and discharge to collection system | 541(61%) | 340(39%) | 881 | ≥ 16 | 108.3 | 4.98 | 0.06 | 4.50 |
| D-J | Route storm drainage directly to bay, store CSO's utilizing available in-system storage and discharge to collection system | 541(100%) | 0(0%) | 541 | ≥ 6 | 57.2 | 12.10 | 0.06 | 9.80 |
| D-K | Screen and disinfect CSO's and separate storm drainage and discharge to bay | 541(61%) | 340(39%) | 881 | < 1 | 1.4 | 3.00 | 0.09 | 3.08 |
| Entire Dorchester Area (BOS-088/089 and BOS-090) | | | | | | | | | |
| D-L | Route CSO's from BOS-090 tributary area to BOS-089 CSO outfall conduit, route Savin Hill storm drainage to bay, store CSO's and remaining BOS-089 storm drainage utilizing off-line storage and discharge to collection system | 867(94%) | 59(6%) | 926 | 1 | 27.6 | 29.25 | 0.10 | 23.67 |
| | | | | | 4 | 56.4 | 21.78 | 0.07 | 17.55 |
| | | | | | 12 | 145.2 | 15.82 | 0.05 | 12.63 |

Notes: (1) Based on June 1979 price levels (ENR 2900) and includes an allowance for engineering and contingencies.
(2) 1980 present worth based on a discount rate of 6-7/8 percent per annum.

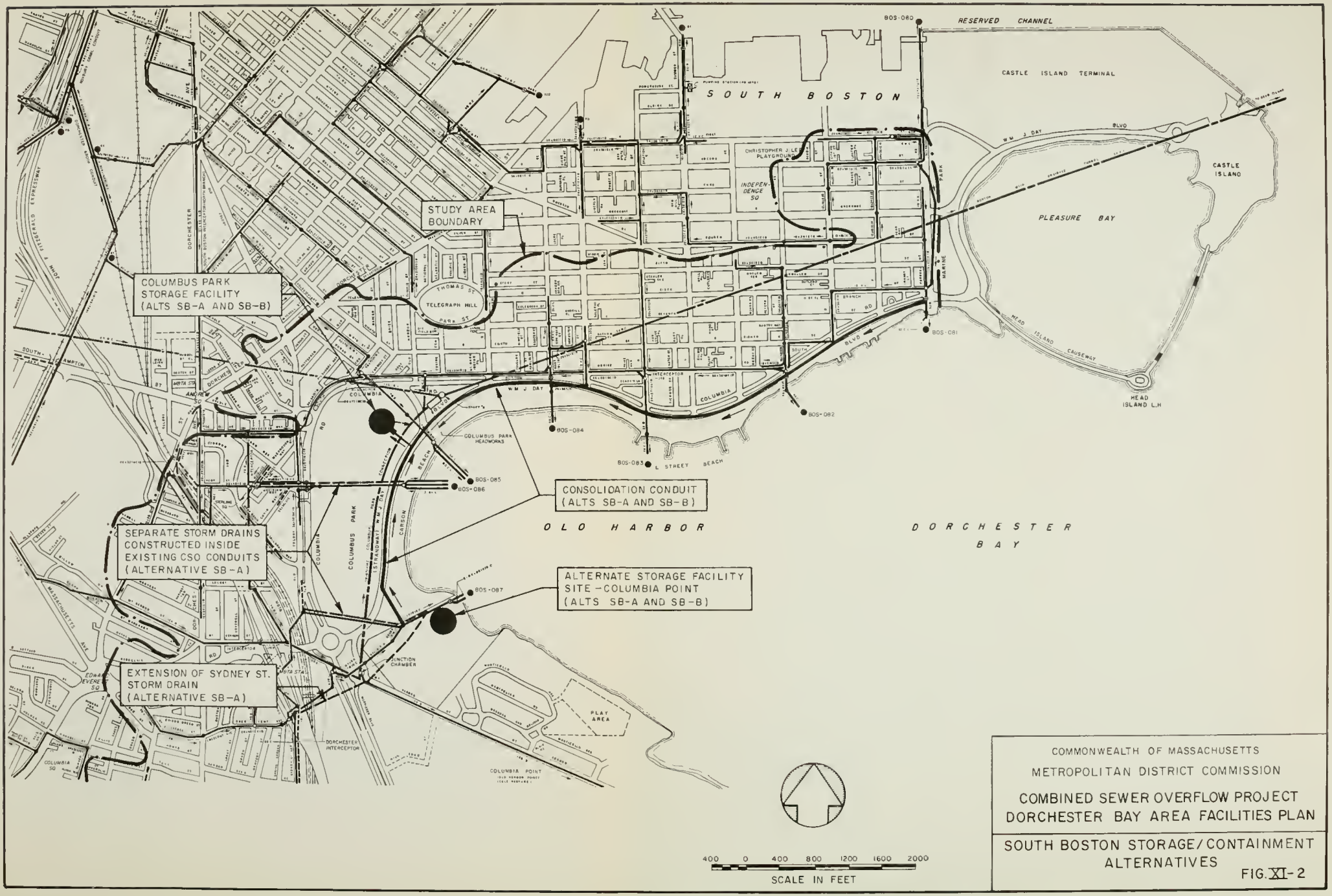
| Control Level (violation days per year) | Approximate Required Storage Volume, mil. gal. Alternative SB-A | Alternative SB-B |
|---|---|---|
| 1 | 11.6 | 14.3 |
| 4 | 7.2 | 8.8 |
| 12 | 3.8 | 4.8 |

The facilities associated with each alternative are shown on Figure XI-2, and include a consolidation conduit and an underground storage tank equipped with screening and pumping facilities. The preferred location of the storage facility is in the northern portion of Columbus Park, with an alternate site located on Columbia Point near the existing Bayside Mall. The centrally located Columbus Park site is preferred for the following reasons:

- o A central storage location reduces the size and length of the southern portion of the consolidation conduit (i.e., the portion between BOS-086 and BOS-087), thus reducing costs.
- o A central storage location reduces the extent of tank excavation since the facilities could be installed at a slightly higher elevation, also reducing costs.
- o The site is owned by the City of Boston, thus the purchase of land would not be required.
- o The proximity of the site to the Columbus Park Headworks would allow utilization of the headworks' maintenance forces.

The alternate site is located near the existing Bayside Mall adjacent to the MDC police station, and consists of parkland owned by the MDC. Under the proposed Columbia Point redevelopment plan by the Boston Redevelopment Authority (BRA) this area will remain as open park land. Final site selection will be determined during the preliminary design phase, and it will be based on further detailed site evaluations, and input from the public. The estimated capital costs of Alternatives SB-A and SB-B (Table IX-1) would be increased by about $1,600,000 if the storage facilities were to be located at the alternate site. A description of the facilities based on the preferred Columbus Park site has been used in this chapter.

As shown of Figure XI-2, a consolidation conduit is required to convey overflows from the seven South Boston CSO outlets to the proposed storage facility. The conduit would begin in Day Boulevard at Farragut Road, and extend west in Day Boulevard to a point approximately 200-ft. beyond the BOS-085 CSO conduit. This northern portion of the consolidation conduit would convey CSO's from areas tributary to CSO outlets BOS-081 through

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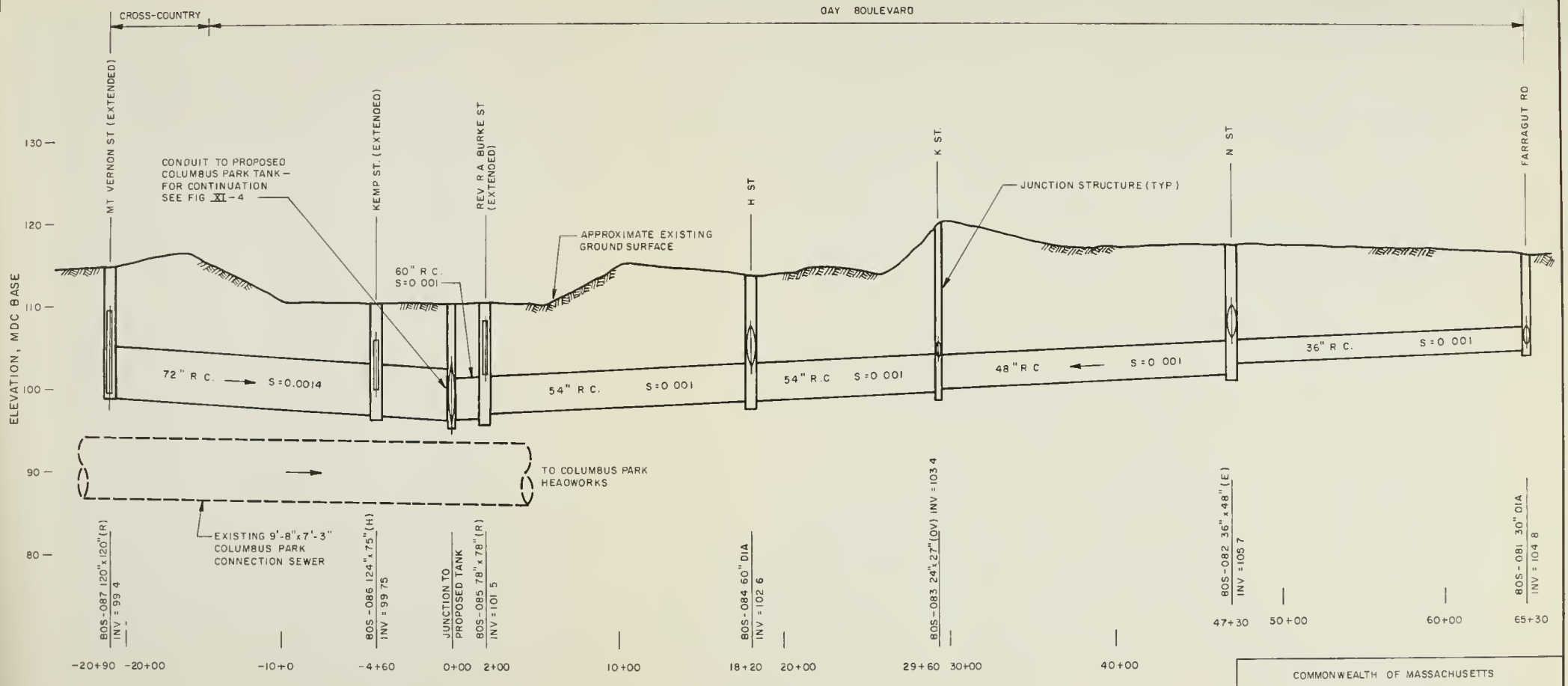
SOUTH BOSTON STORAGE/CONTAINMENT ALTERNATIVES

FIG. XI-2

BOS-085 to the storage facility. Flows from areas tributary to BOS-086 and BOS-087 would be conveyed to the storage facility by the southern portion of the consolidation conduit which would begin at the BOS-087 CSO outlet and extend north, generally located in Day Boulevard, to the junction with the northern portion of the conduit. The approximate total length of the conduit is 8,600 ft. and its size varies over the length, and depends upon the control level. A profile of the conduit required under Alternative SB-B for the 4 overflow days per year control level is shown on Figure XI-3.

From the junction of the northern and southern portions of the conduit, wet weather flows would be conveyed through an 84-in diameter conduit to a screen house where it would pass through automatically cleaned bar screens before it entered the storage tank. A preliminary layout of these facilities is shown on Figure XI-4. The sizes shown correspond to the 4 overflow days per year control level for Alternative SB-B (i.e., approximate tank size = 8.8 mil. gal.) which is the most cost-effective South Boston storage/containment alternative, as will be discussed later in this section. For the cost effective analysis, it has been assumed that a circular, reinforced concrete tank would be utilized for storage. A preliminary cost comparison indicated that a rectangular tank with compartments would cost approximately the same as a comparably sized circular tank. Accordingly, each type of tank will be evaluated in more detail and a final tank configuration will be selected during the preliminary design phase. As shown on Figure XI-4, the tank facilities would be equipped with a flushing system and a conical base slab with a central sump to facilitate settled solids removal. The stored flow would be pumped from the tank to the Columbus Park connection over a 12-hour period, thus the pump units, along with a standby unit, were sized accordingly. The actual allowable dewatering rate will be based on the total storage volume from all four project areas as compared to the available excess treatment capacity at the Deer Island treatment plant, and will be determined during the preliminary design phase. As indicated on Figure XI-4, the storage tank would be located entirely below ground. A superstructure would be required to house the screens, pumps, odor control equipment, screening hoppers and controls, and to allow truck access for screenings removal. Tank sizes ranged from 160-ft. diameter and 25-ft. deep for Alternative SB-A at the 12 overflow days per year control level (Volume = 3.8 mil. gal.), to 300-ft. diameter and 26-ft. deep for Alternative SB-B at the 1 overflow day per year control level (Volume = 14.3 mil. gal.). The size of the superstructure would be approximately 30-ft. x 30-ft. and 12 to 15-ft. high. The facility would be manned continuously with additional personnel required during and immediately after an overflow occurrence.

Hydraulic relief of the consolidation conduit and storage facility would be provided at the existing CSO outlets. Stationary bar racks would be provided on these overflows to prevent the discharge of large floatables to the bay during those storm events that exceed the capacity of the proposed facilities. On those instances when the capacity is exceeded (i.e., 1, 4 or 12 days per year depending on the control level), the quality of the overflow would, most likely, be somewhat higher than the volume that entered the storage facility since the effects of the first flush would be over.




NOTE · REFER TO FIG. XI-2 FOR A PLAN VIEW OF THIS CONDUIT

SCALE IN FEET

0 10 20
VERTICAL

0 500 1000
HORIZONTAL

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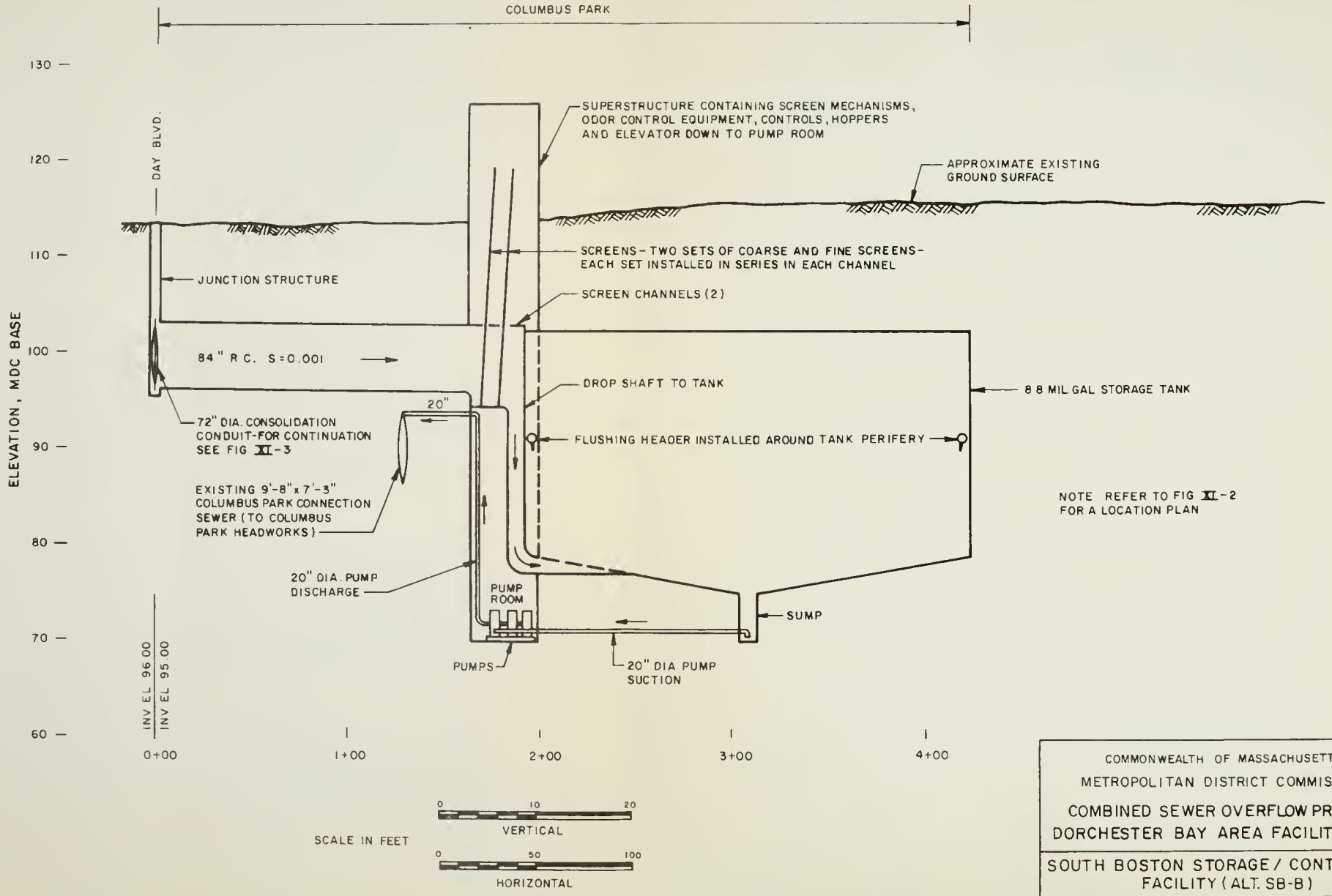
PROFILE OF PROPOSED SOUTH BOSTON CONSOLIDATION CONDUIT
FIG. XI-3




COMMONWEALTH OF MASSACHUSETTS

METROPOLITAN DISTRICT COMMISSION

COMBINED SEWER OVERFLOW PROJECT

DORCHESTER BAY AREA FACILITIES PLAN

SOUTH BOSTON STORAGE / CONTAINMENT FACILITY (ALT. SB-B)

FIG. XI-4

As discussed earlier, under Alternative SB-A the runoff generated from the 59 acres of separate sewered areas (Figure XI-1), would be conveyed directly to the bay through new conduits, thus reducing the required size of the storage facility. As shown on Figure XI-2, three new storm drains would be required to intercept the three existing storm drains that serve this area, and bypass the runoff around the proposed consolidation conduit. The new storm drains would be constructed either in a separate trench or located inside of the existing CSO conduits where possible.

The estimated costs of Alternatives SB-A and SB-B for each control level are shown on Table XI-1. Also shown is the estimated annual total coliform load to the bay from the South Boston tributary area expressed in pseudopounds (i.e., mil. gal. x coliform concentration in no. per 100 ml x 8.34). Shown on Figure XI-5 is a plot of the estimated 1980 present worth cost vs. the number of CSO violation days per year for the two storage alternatives (Alternative SB-C which consists of screening and disinfection, will be discussed later in this chapter). Examination of Figure XI-5 reveals that no alternative is clearly more cost-effective than the others, and the present worth costs are within 10 percent of each other.

Dorchester - Fox Point. Three storage/containment alternatives were developed and evaluated for the Fox Point portion of the study area (Figure XI-1), and are designated herein as D-A, D-B, and D-C. Each involves the storage - pumpback concept, with the differences in alternatives being the type of storage utilized (i.e., existing in-line, new off-line, or combinations of both). Alternative D-A involves the use of an off-line storage facility; D-B includes the use of available existing in-system storage to reduce the required off-line tank volume; and D-C consists of using only the available in-system storage volume. Each alternative includes routing the storm runoff from the Savin Hill area directly to the bay, since this proved to be cost effective in all instances as compared to storage of the runoff.

The facilities associated with Alternatives D-A and D-B were sized and costs estimated for the control levels of 1, 4 and 12 violation days per year. Since Alternative D-C involves the utilization of available, existing in-line storage, only the control level that results from utilization of this finite volume was investigated. This resulting control level was determined from the storage-treatment curves developed from the model SEMSTORM. The required storage volumes for each control level for Alternatives D-A and D-B were also based on the storage-treatment curves that were refined as discussed previously for the South Boston alternatives. A discussion of each alternative follows.

Alternative D-A. This alternative involves the use of off-line storage to reduce the annual number of CSO violation days. Based on the results of the model SEMSTORM and utilizing the refined storage-treatment curves developed for the Fox Point tributary area (assuming the Savin Hill storm drain is re-routed directly to the bay), the storage volumes required to achieve the selected control levels are as follows:

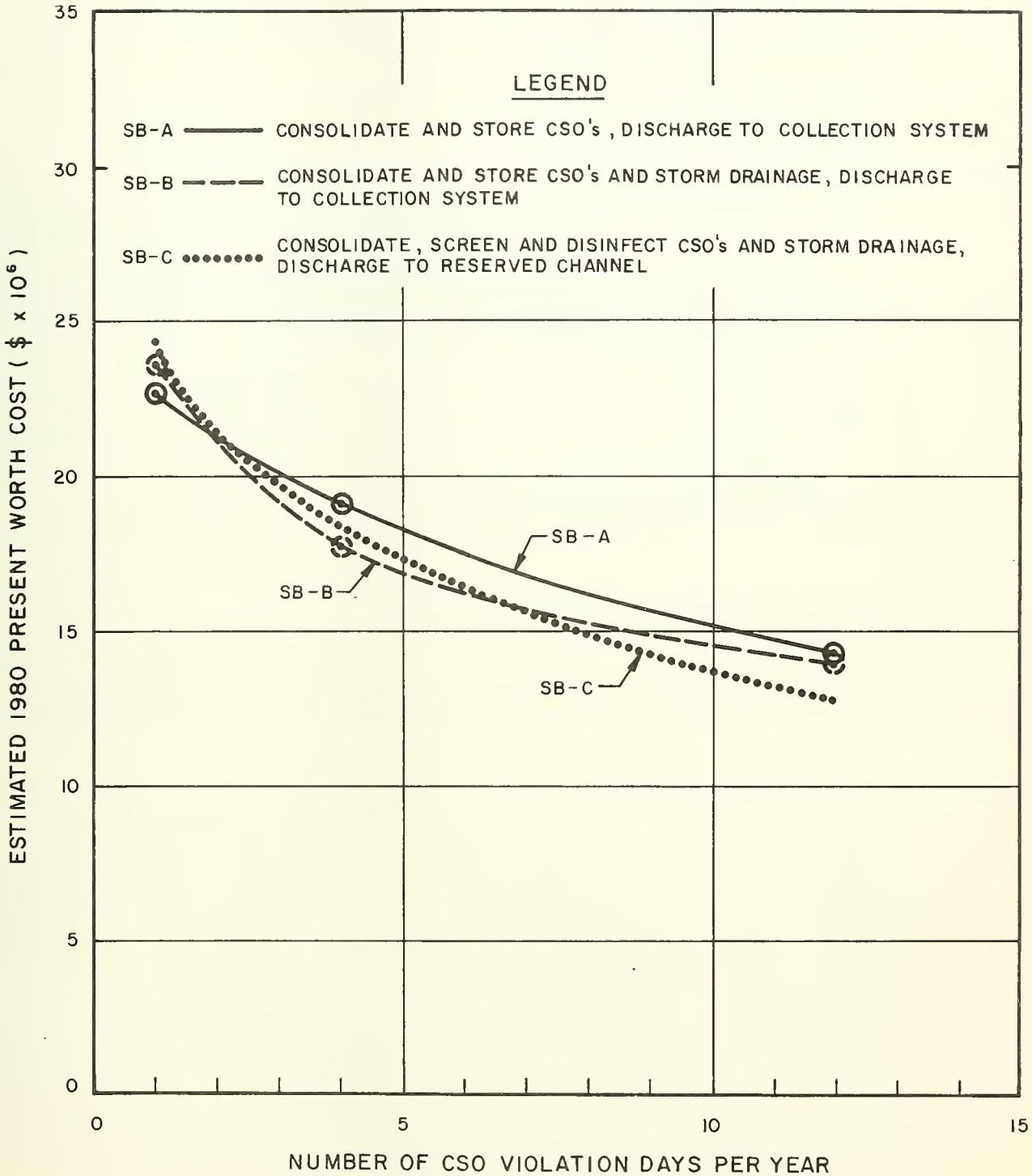



FIG. XI-5 EVALUATION OF SELECTED ALTERNATIVES FOR SOUTH BOSTON

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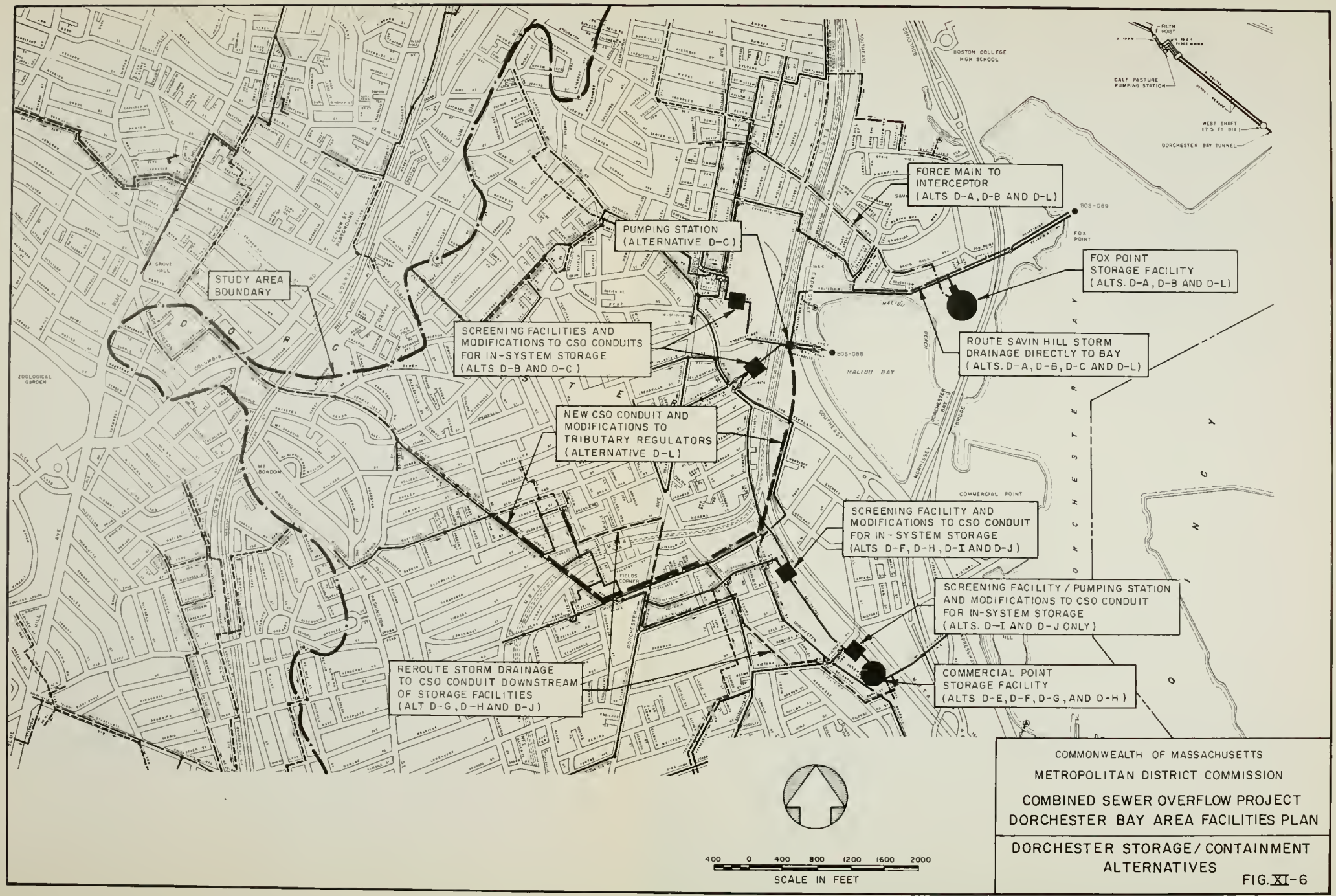
COMBINED SEWER OVERFLOW PROJECT
DORCHESTER BAY AREA FACILITIES PLAN

| Control Level (Violation Days per Year) | Approximate Required Storage Volume (mil. gal.) |
|---|---|
| 1 | 5.3 |
| 4 | 2.5 |
| 12 | 1.0 |

As mentioned earlier, a cost comparison indicated that it is more cost-effective to route the runoff generated from the 40 acre separate sewered area located in the southern portion of Savin Hill, directly to the bay. This area, shown on Figure XI-1, is tributary to the 30-in. storm drain that discharges into the 96-in. x 120-in. CSO conduit leading to BOS-089. If the runoff from this area were stored, the required volumes would increase by 0.6, 0.3 and 0.2 mil. gal. for the 1, 4 and 12 days per year control levels, respectively. The associated increases in the estimated capital costs of the storage facilities would range from $900,000 (12 days/year) to $1,300,000 (1 day/year) as compared to $350,000 which is the estimated capital cost of re-routing the Savin Hill storm drain. As shown on Figure XI-6, the 30-in. drain would be intercepted at the existing CSO conduit and extended parallel to the conduit, and reconnected downstream of the storage facility.

As shown on Figure XI-1, there are three other small, separate sewered areas within the Fox Point tributary area totaling about 59 acres. Since they are located at the extremity of the area, it would not be feasible to route runoff from these areas directly to the bay considering the length of the required conduits, thus they are considered part of the total area tributary to the storage facilities (i.e., 385 acres).

The off-line storage facilities required for the Fox Point alternatives would be essentially the same as those described for the South Boston alternatives and shown on Figure IX-4. They would include a superstructure that would contain screens, dewatering pumps, odor control equipment, screening hoppers and controls. The off-line storage tanks (D-A and D-B) were sized and construction costs estimated based on a circular, reinforced concrete tank that is entirely below ground. The selected site used is located on the east side of Malibu Beach in the existing MDC owned parking lot, as shown on Figure XI-6. This site was selected over the privately owned site located off of Freeport Street between Freeport Way and the MBTA tracks (refer to Figure X-6 in Chapter X) primarily due to space limitations and potential land acquisition problems. The loss of potential property tax revenues by utilizing the privately owned site was also considered. Construction of the facilities at the selected site would have relatively minor impact on beach activities with the major disadvantage being the temporary disruption to the parking lot.


STUDY AREA BOUNDARY
SCREENING FACILITIES AND MODIFICATIONS TO CSO CONDUITS FOR IN-SYSTEM STORAGE (ALTS D-B AND D-C)
PUMPING STATION (ALTERNATIVE D-C)
NEW CSO CONDUIT AND MODIFICATIONS TO TRIBUTARY REGULATORS (ALTERNATIVE D-L)
REROUTE STORM DRAINAGE TO CSO CONDUIT DOWNSTREAM OF STORAGE FACILITIES (ALT D-G, D-H AND D-J)
FORCE MAIN TO INTERCEPTOR (ALTS D-A, D-B AND D-L)
FOX POINT STORAGE FACILITY (ALTS. D-A, D-B AND D-L)
ROUTE SAVIN HILL STORM DRAINAGE DIRECTLY TO BAY (ALTS. D-A, D-B, D-C AND D-L)
SCREENING FACILITY AND MODIFICATIONS TO CSO CONDUIT FOR IN-SYSTEM STORAGE (ALTS D-F, D-H, D-I AND D-J)
SCREENING FACILITY / PUMPING STATION AND MODIFICATIONS TO CSO CONDUIT FOR IN-SYSTEM STORAGE (ALTS. D-I AND D-J ONLY)
COMMERCIAL POINT STORAGE FACILITY (ALTS D-E, D-F, D-G, AND D-H)
BOS-089
BOS-088
FOX POINT
MALIBU BEACH
MALIBU BAY
COMMERCIAL POINT
BOSTON COLLEGE HIGH SCHOOL
CALF PASTURE PUMPING STATION
WEST SHAFT 17.5 FT DIA
DORCHESTER BAY TUNNEL
DORCHESTER BAY
ZOOLOGICAL GARDEN
MT BOWDOIN
GROVE HALL
FIELDS CORNER
400 0 400 800 1200 1600 2000
SCALE IN FEET

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METROPOLITAN DISTRICT COMMISSION

COMBINED SEWER OVERFLOW PROJECT

DORCHESTER BAY AREA FACILITIES PLAN

DORCHESTER STORAGE/ CONTAINMENT ALTERNATIVES

FIG. XI-6

Flow diversion into the storage facility, as well as hydraulic relief should the storage capacity be exceeded, would be provided by an automatically controlled gate installed in the existing CSO conduit. Similar to the South Boston storage facility, dewatering of the tank would be accomplished by pumping. The storage volume would be pumped from the tank via force main (a possible route is shown on Figure XI-6) to the Dorchester interceptor. The pumps were sized based on the maximum available interceptor capacity which was assumed for this analysis to be the maximum allowable tank dewatering rate. Tank sizes ranged from 100 ft. diameter and 20-ft. deep for the 12 days per year control level (Volume = 1.0 mil. gal.) to 190-ft. diameter and 25-ft. deep for the 1 day per year control level (volume = 5.3 mil. gal.) The operation and general configuration of the Fox Point storage facilities would be approximately the same as those for South Boston (Figure XI-4). The estimated capital, operation and maintenance and 1980 present worth costs for Alternative D-A are shown on Table XI-1 for each control level.

Alternative D-B. Alternative D-B contains all of the elements of D-A, except available existing in-system storage is utilized in conjunction with off-line storage to reduce the annual number of CSO violation days. Referring to Figure XI-6, the CSO conduit that conveys overflows to outlet BOS-089 would be utilized for in-system storage. It begins at Auckland Street as a 144-in. x 132-in. horseshoe-shaped sewer and extends generally west to the overflow weir chamber near the Southeast Expressway. From this point it continues as a 96-in. x 120-in. rectangular conduit north and then east around Malibu Beach to the tide gate chambers located on the west side of Morrissey Boulevard. The overflow weir chamber located on this conduit behind the Dorchester Yacht Club was described in Chapter IV, and it serves to provide hydraulic relief to outlet BOS-089 through the BOS-088 outlet which discharges to Malibu Bay. The 84-in. diameter CSO conduit extending from Freeport St. to the MBTA tracks would also be utilized for storage. These conduits provide the only usable existing in-system storage volume within the Fox Point tributary area, since other tributary CSO conduits are significantly smaller in size and their utilization would cause surcharging and possibly flooding conditions.

The available storage volume in the CSO conduits was based on allowing the conduits to fill to an elevation corresponding to the elevation of the lowest regulator connection, in order to prevent surcharge conditions in tributary sewers. This resulted in an available storage volume of about 1.4 mil. gal. Utilization of this volume would result in the following required off-line storage volumes for each control level:

| | Storage Volumes, mil. gal. | | |
|---|---|---|---|
| Control Level (Violation Days/Yr) | Total Required | In-System | Off-Line |
| 1 | 5.3 | 1.4 | 3.9 |
| 4 | 2.5 | 1.4 | 1.1 |
| 12 | 1.0 | 1.4 | 0.0 |

The 12 days/year control level would not require an off-line facility. In fact, utilization of the 1.4 mil. gal. of in-system storage results in an estimated control level of about 9 violation days per year. This concept comprises Alternative D-C and will be discussed in the next section.

The facilities required to utilize the in-system storage available in the CSO conduits are shown on Figure XI-6. Two screening facilities would be required, each located on currently vacant, privately owned land at the upstream end of the two CSO conduits. Each facility would include coarse (1-1/2" bar spacing) and fine (3/4" bar spacing) automatically cleaned bar screens located in series in the conduit with a superstructure above containing the screen controls, screenings hoppers and other related equipment. A bypass conduit would be provided at each screening facility to allow continuous operation during construction and also to serve as an emergency relief in the event of screening equipment malfunction. The off-line facility would be utilized as primary storage, with the existing conduit utilized for secondary storage once the capacity of the off-line facility has been exceeded. A downstream control gate located near the existing tide gate chamber of the east side of Morrissay Boulevard would serve to divert flow into the off-line storage facility, and also would allow flow to be stored in the CSO conduit. If the total storage volume is exceeded, the control gate would operate automatically to allow hydraulic relief to occur through the BOS-089 outlet. A flushing system would be installed in the CSO conduit that would serve to convey settled solids to the storage tank after the overflow event. All stored flow would be pumped from the tank to the Dorchester interceptor, as discussed previously. Off-line tank physical sizes would be about 100-ft. diameter and 22-ft. deep for the 4 days per year control level (Volume = 1.1 mil. gal.) and 160-ft. diameter and 26-ft. deep for the 1 day per year level (volume = 3.9 mil. gal.). The tank facilities would be similar to those described for the South Boston alternatives (Figure XI-4), however no screens would be required since screening would be performed at the upstream screening facilities. Accordingly, a much smaller superstructure would be necessary to house the pumps, odor control equipment, controls and to provide tank access.

The estimated capital, operation and maintenance and 1980 present worth costs for Alternative D-B are shown on Table XI-1 for each control level.

Alternative D-C. This alternative involves the utilization of the existing available in-system storage volume of 1.4 mil. gal. to reduce the estimated number of CSO violation days to about 9 per year.

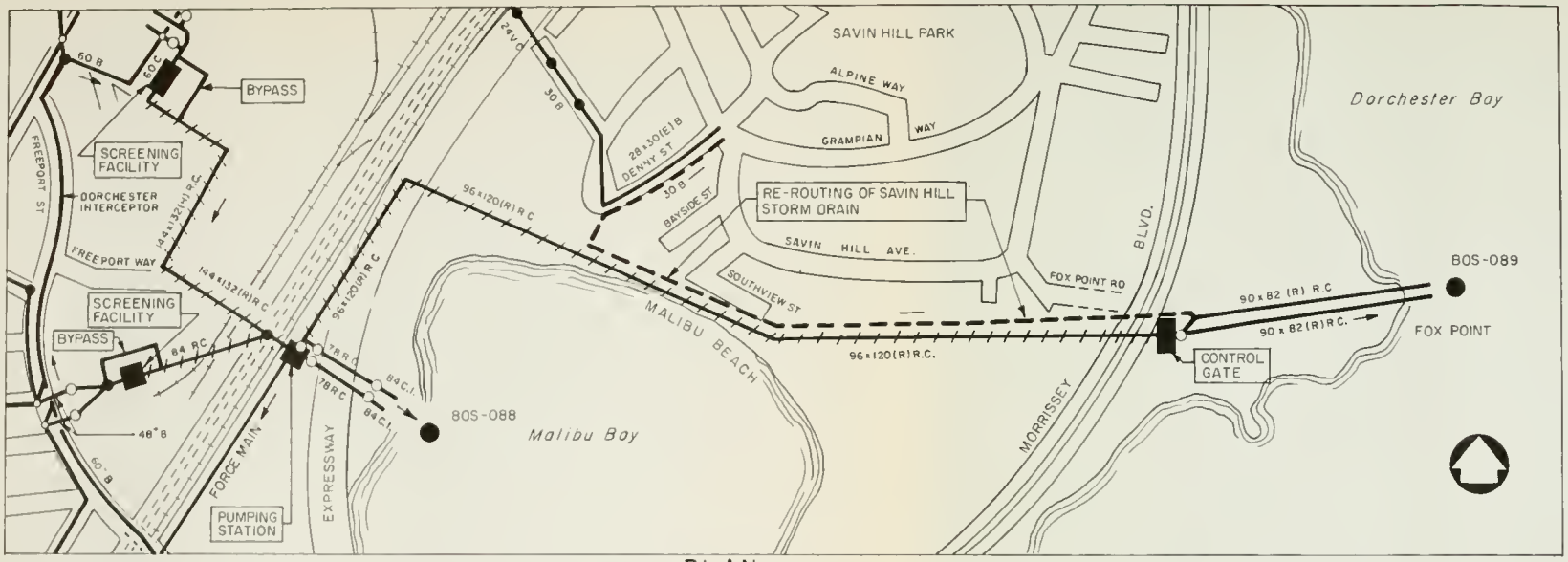
Required facilities include two screening facilities, each located at the upstream end of the two CSO conduits as in Alternate D-B. A downstream control gate located at the existing tide gate chamber near Morrissey Boulevard would also be required under this alternative, for the purpose of controling the storage volume in the conduit, and also to provide hydraulic relief in the event the storage volume is exceeded. A pumping station, possibly located on state owned land at the diversion weir structure near the Southeast Expressway (Figure XI-6) would serve to pump the contents of the conduits to the Dorchester interceptor. An alternate location for the pumping station would be on a state-owned, vacant site located between the expressway and Malibu Beach, just south of McConnell Park.

As described under Alternate D-B. a flushing system would be required in the CSO conduits to convey settled solids to the pumping station. A header pipe, installed on the interior crown of the conduits and equipped with spray nozzles, would utilize potable city water for flushing purposes. Since the CSO conduit presently slopes towards the BOS-089 outlet, the portion of the conduit located downstream of the proposed pumping station would require a new invert sloped towards the pumping station.

Details of the facilities associated with this alternative are shown on Figure XI-7. The estimated capital, operation and maintenance and 1980 present worth costs are shown on Table XI-1.

Comparison of Fox Point Storage/Containment Alternatives. A summary of pertinent information associated with the three Fox Point storage/containment alternatives is shown on Table XI-1. Shown on Figure XI-8 is a plot of the estimated 1980 present worth cost vs. the estimated annual number of CSO violation days for each alternative (Alternative D-D which consists of screening and disinfection will be discussed later in this chapter). This information indicates that the estimated annual number of CSO violation days can be reduced from about 24 (assuming the Hoyt Street regulator is modified) to about 9 by using only existing, available in-system storage (Alternative D-C) at a total cost of about $5.32 million. This results in a unit cost of about $0.3 million per violation day reduction. To increase the control level to about 4 violation days per year (i.e., one per bathing season) would require the construction of new, off-line storage facilities and an additional expenditure (present worth) of about $3.00 million (total cost of $8.32 million - Alternative D-A which is less costly then Alt. D-B at this control level). The unit cost to achieve this additional control is about $0.60 million per violation day reduction, or 40 percent greater than Alternative D-C. Alternative D-C has an additional advantage since it represents the maximum utilization of the existing system for CSO abatement. The adverse environmental impacts during and after construction would also be minimized, since utilization of the Malibu Beach parking lot for construction of a storage tank and a permanent superstructure (i.e., Alternatives D-A and D-B) would not be required. The screening and pumping facilities under Alternative D-C would be relatively small structures located such that the impacts during and after construction would be minimal.

Dorchester - Commercial Point. Six storage/containment alternatives were developed and evaluated for the Commercial Point portion of the study area (Figure XI-1), and are designated herein as D-E, D-F, D-G, D-H, D-I, and D-J. Similar to the Fox Point alternatives, each involves the storage-pumpback concept with the differences in alternatives being the use of existing in-system storage and/or the routing of runoff generated from the separate sewered areas. Facilities associated with Alternatives D-E, F, G and H were sized and costs estimated for the control levels of 1, 4 and 12 violation days per year. Alternatives D-I and D-J utilize existing, available in-system storage only, thus only the control level that results form utilization of this finite volume was investigated. Similar to the South Boston and Fox Point alternatives, the required




PLAN

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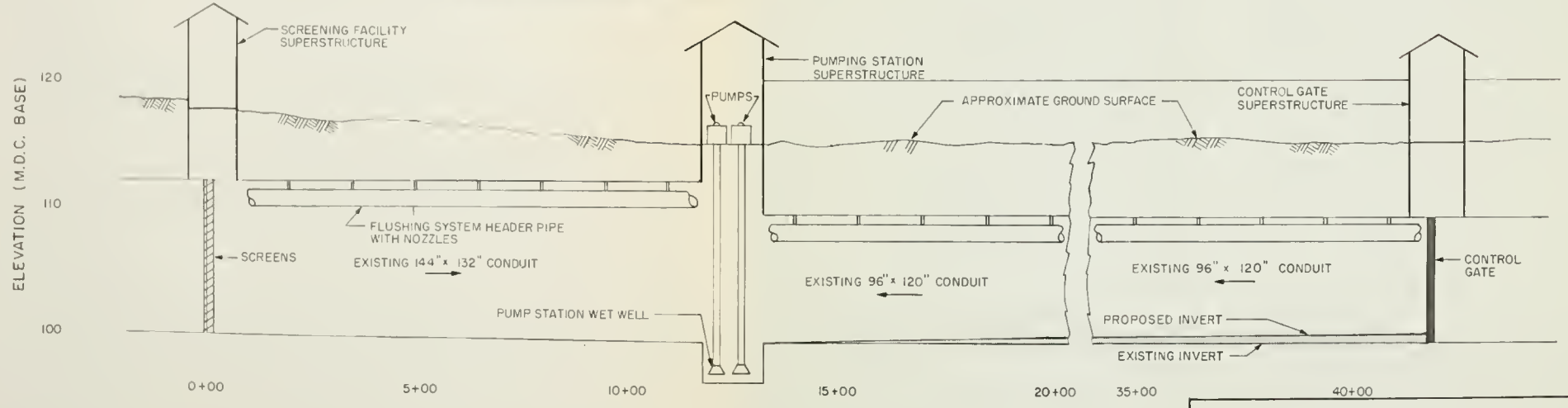


SECTION

SCALE IN FEET

0 10

VERTICAL

100 0 100 200 300 400

HORIZONTAL

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COMBINED SEWER OVERFLOW PROJECT

DORCHESTER BAY AREA FACILITIES PLAN

FOX POINT SELECTED STORAGE/CONTAINMENT ALTERNATIVE (D-C)

FIG. XI-7

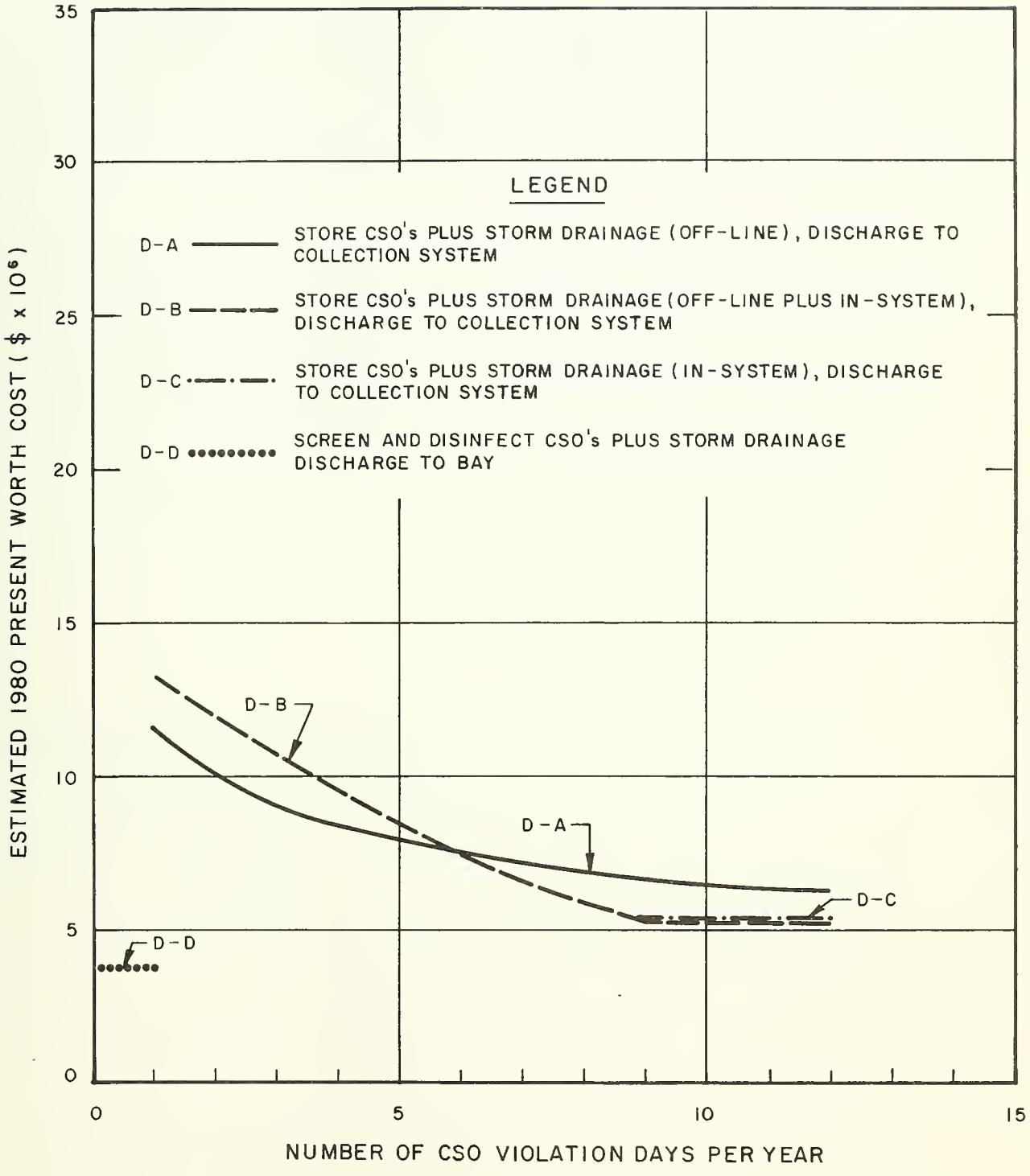


FIG. XI-8 EVALUATION OF SELECTED ALTERNATIVES FOR DORCHESTER AREA TRIBUTARY TO BOS-088/089 (FOX POINT)

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METROPOLITAN DISTRICT COMMISSION
COMBINED SEWER OVERFLOW PROJECT
DORCHESTER BAY AREA FACILITIES PLAN

storage volumes at each control level for Alternatives D-E, F, G, and H and the resulting control level from using in-system storage for Alternatives D-I and D-J were determined from the storage-treatment curves developed from the model SEMSTORM. Curves representing the combined sewered areas only, as well as combined plus separate sewered areas were developed and refined within the 0 to 12 violation days per year range. The facilities required for each alternative are shown on Figure XI-6, and Table XI-1 summarizes pertinent information associated with each. A discussion of each alternative follows.

Alternative D-E. This alternative involves the use of off-line storage to reduce the annual number of CSO violation days. As shown on Table XI-1, the area tributary to the off-line facilities would include both combined and separate sewered areas resulting in a total tributary area of about 881 acres (these areas are shown on Figure XI-1). Based on the results of the model SEMSTORM, and utilizing the refined storage-treatment curves for the entire tributary area, the storage volume required to achieve the selected control levels are as follows:

| Control Level (Violation days per year) | Required Storage Volume (mil. gal.) |
|---|---|
| 1 | 12.5 |
| 4 | 6.4 |
| 12 | 2.8 |

As shown on Figure XI-6, the facilities required for this alternative include an off-line storage facility located at McMorrow Playground, which is bounded by Victory Road, Houghton Street and the MBTA tracks. This site, owned by the MDC and having an area of approximately 12 acres, represents the only feasible location for a storage facility within the Commercial Point tributary area. About 3 to 4 acres of the northern portion of the playground would be temporarily disrupted during construction, with only a superstructure (approximate size would be about 35 ft. x 35 ft. and 12-15 ft. high) remaining at completion of construction. Odor control facilities would be incorporated into the design of the facilities. An alternate site located at the BOS-090 outlet south of Commercial Point was also considered, however, construction difficulties due to poor subsurface soil conditions and dewatering problems precludes locating a major structural facility at this location.

The facilities and their operation would be essentially the same as described for South Boston Alternatives SB-A and B, and Fox Point Alternative D-A. Included would be a superstructure containing the screens, dewatering pumps, odor control equipment, hoppers and controls. The storage tank, located entirely below ground, was sized and construction costs estimated based on a circular, reinforced concrete structure of the same configuration as the tank shown on Figure XI-4. Tank sizes ranged from 140-ft. diameter and 24-ft. deep for the 12-days per year control level (volume = 2.8 mil. gal.), to two, 180-ft. diameter and 33-ft. deep

tanks for the 1 day per year level (total volume of both tanks = 12.5 mil. gal.). A bypass conduit containing an automatically controlled gate would be required for hydraulic relief of the facility in the event the storage volume is exceeded. Similar to the Fox Point alternatives, stored flow would be pumped to the Dorchester interceptor as capacity became available. Estimated costs and coliform loads to the bay for each control level is shown in Table XI-1.

Alternative D-F. Alternative D-F is similar in concept to Alternative D-B (Fox Point) in that existing, available in-system storage is utilized in conjunction with off-line storage to reduce the annual number of CSO violation days. Referring to Figure XI-6, a portion of the CSO conduit that conveys overflows to outlet BOS-090, and beginning at Christopher Street (extended) near the MBTA tracks, and extending to the tide gate chamber located between Freeport Street and the Southwest Expressway would be utilized for storage. Its size ranges from a 168-in. x 138-in. (horseshoe-shaped) to a 144-in. x 144-in. (horseshoe-shaped). This conduit provides the only useable existing in-system storage volume within the tributary area for the same reasons discussed under Alternative D-B (Fox Point). The available storage volume is estimated to be about 2.0 mil. gal., and was based on allowing the conduit to fill to an elevation corresponding to the elevation of the lowest regulator connection, in order to prevent surcharge, and possibly flooding conditions in tributary sewers. Utilization of this volume would require off-line tank volumes as follows:

| | Storage Volumes, mil. gal. | | |
|---|---|---|---|
| Control Level (Violation Days/Year) | Total Required | In-System | Off-Line |
| 1 | 12.5 | 2.0 | 10.5 |
| 4 | 6.4 | 2.0 | 4.4 |
| 12 | 2.8 | 2.0 | 0.8 |

The facilities associated with this alternative are shown on Figure XI-6. They are essentially the same as those required for Alternative D-E, however, a screening facility would be required at the upstream end of the CSO conduit for floatables and large solids removal. This facility would be located on privately owned land currently zoned industrial, and would essentially be the same as the screening facilities described for Alternative D-C and shown on Figure XI-7. A downstream control gate located at the existing tide gate chamber would also be required to provide storage in the conduit and hydraulic relief in the event the total storage volume is exceeded. A flushing system in the conduit as well as in the tank would be required for settled solids removal as described under Alternative D-C. In order to allow the flushing water to flow to the tank for eventual pumping to the Dorchester interceptor, the invert

of the portion of the conduit located downstream of the tank would require modification in order to provide a slope towards the tank. Under this alternative, CSO's would enter the off-line facility first, with the conduit utilized only if the off-line tank volume is exceeded. Off-line tank sizes ranged from 100-ft. diameter and 14-ft. deep for the 12-day per year control level (volume = 0.8 mil. gal.), to two, 200-ft. diameter and 20-ft. deep tanks for the 1 day per year level (total storage volume = 10.5 mil. gal.).

Estimated costs for each control level under this alternative are given in Table XI-1. As shown, the costs are similar to those of Alternative D-E, which means that the cost associated with utilizing the 2.0 mil. gal. of in-system storage is about equal to the cost of providing an additional 2.0 mil. gal. of off-line storage at the McMorrow Playground site. Since the tributary area served is the same for each alternative (881 acres), the estimated annual total coliform load to the bay is also the same for each alternative.

Alternative D-G. This alternative includes all the elements of Alternative D-E (utilization of off-line storage only), however, storage would be provided for only the combined sewered areas (541 acres) with the runoff from the separate sewered areas (340 acres) routed directly to the bay. As shown on Figure XI-6, two new storm drains would be required to intercept the two existing storm drains serving the separated areas, and runoff would be conveyed to the existing BOS-090 CSO conduit at a point located downstream of the storage facilities. The northern separated area would require a 54-in. diameter storm drain beginning at the 54-in. drain in Charles Street and generally following a route shown on Figure XI-6 to a junction with the existing 72-in. x 72-in. drain at Gibson Street, which drains the southern separated area. From this point, a single 120-in. diameter drain would convey runoff from both separate sewered areas for discharge to the existing CSO conduit downstream of the storage facilities. The route shown on Figure XI-6 is considered preliminary, and does not reflect the results of a detailed subsurface (i.e., utilities) investigation, the results of which may possibly require the conduit to be located in an easement. Estimated costs, were based on the route shown on Figure XI-6.

Routing the runoff from the 340 acres of separate sewered areas directly to bay reduces the area tributary to the storage facilities from 881 acres to about 541 acres. This results in the following required storage volumes to achieve the selected CSO control levels:

| Control Level (Violation days per year) | Required Storage Volume (mil. gal.) |
|---|---|
| 1 | 6.3 |
| 4 | 2.8 |
| 12 | 0.9 |

The off-line storage facilities located in McMorrow Playground would be the same as those described for Alternative D-E. The tank sizes for this alternative ranged from 100-ft. diameter and 16-ft. deep for the 12 day per year control level (volume = 0.9 mil. gal.) to 180 ft. diameter and 33-ft. deep for the 1 day per year level (volume = 6.3 mil. gal.).

Estimated costs and coliform loads to the bay for each control level under this alternative area shown on Table XI-1. The increased annual total coliform load to the bay, as compared to Alternative D-E, is reflective of the reduced tributary area served, since runoff from about 341 acres having an average total coliform concentration of about $2 \times 10^6$ MPN per 100 ml. would discharge directly to the bay. Comparison of estimated costs between Alternatives D-E and D-G indicates that it is less costly to route the storm drainage to the bay than to provide the additional 6.2 mil. gal. of off-line storage volume required for the separated areas, for the 1 day per year control level. For the 4 and 12 days per year levels, however, it would be more cost-effective to provide storage for the runoff than to route it to the bay. This is indicative of a greater economy-of-scale for the storage tanks than for the new storm drains, which appears reasonable considering that installation costs of the new drains would not be reduced significantly with reduced pipe sizes.

Alternative D-H. This alternative is a combination of Alternatives D-F and D-J, since it utilizes in-system storage as well as off-line storage for control of CSO's generated from the combined sewered areas only. The runoff from the 340 acres of separate sewered areas would be routed directly to the bay. The off-line storage facilities would be the same as those described for Alternative D-F, however, since the separate sewered area runoff would not be tributary to the facility, the required off-line storage volumes would be reduces as follows:

| | Storage Volumes, mil. gal. | | |
|---|---|---|---|
| Control Level (Violation Days per year) | Total Required | In-System | Off-line |
| 1 | 6.3 | 2.0 | 4.3 |
| 4 | 2.8 | 2.0 | 0.8 |
| 12 | 0.9 | 2.0 | 0.0 |

Off-line storage facilities would not be required for the 12 day per year control level. Utilization of only the available in-system storage and routing the separate sewered area drainage to the bay would result in a control level of about 6 violation days per year. This concept comprises Alternative D-J and will be discussed later in this chapter.

The required in-system and off-line storage facilities for this Alternative (D-H) would be the same as those described under Alternative D-F. The tank sizes, however, would be smaller ranging from 100-ft. diameter and 14-ft. deep for the 4 violation days per year control level (Volume = 0.8 mil. gal.), to 170-ft diameter and 25-ft deep for the 1 day per year level (Volume = 4.3 mil. gal.). The new storm drains required to route the separate sewered area runoff to the bay would be the same as described for Alternative D-G.

Table XI-1 shows the estimated costs and coliform loads to the bay for each control level. As shown, the costs for the 1 and 4 days per year control levels are similar to Alternative D-G costs, which is again indicative of the relative indifference in costs between utilization of in-system storage vs. providing additional off-line storage. The costs for the 12 days per year level is reflective of the utilization of in-system storage only (i.e., no off-line storage required) with a control level of 6 days per year possible with its full utilization.

Alternative D-I. Alternative D-I involves the utilization of only the existing, available in-system storage volume of 2.0 mil. gal. for reducing the number of CSO violation days per year. The total service area for this alternative includes both the combined plus separate sewered areas, for a total of 881 acres. Based on the storage-treatment curve developed for this area by employing the model SEMSTORM, utilization of the in-system storage volume would result in a control level of about 16 violation days per year (reduces from about 24 per year assuming the Hoyt Street regulator is modified).

Facilities associated with this alternative (Figure XI-6) are essentially the same as those described for Alternative D-F, except that a screening facility/pumping station would be installed at the McMorrow Playground site in place of the off-line storage tank. This screening facility would be required in addition to the screening facility located at the upstream end of the conduit, in order to screen CSO's emanating from the Victory Road overflow conduit. The facility would also include pumps for conveying the stored flow to the Dorchester interceptor as capacity became available. As described for Alternative D-F, a flushing system would be required in the conduit to convey settled solids to the pumping station. Invert modifications to the downstream portion of the conduit would also be necessary to allow gravity flow to the pumping station.

Referring to Table XI-1, the estimated 1980 present worth cost of this alternative is about $4.50 million, which appears to be relatively inexpensive when compared to the other alternatives. However, a maximum control level of only 16 violation days per year would be possible under Alternate D-I.

Alternative D-J. This alternative is identical to Alternative D-I, except that the runoff from the 340 acres of separate sewered areas would be routed directly to the bay, as described under Alternative D-G. Since the area tributary to the storage conduit would be reduced from 881 acres to 451 acres, the control level would be increased from 16 violation days

per year to about 6 days per year. Table XI-1 shows that the estimated annual total coliform load to the bay would be reduced by almost 50 percent by providing this additional control for only the combined sewered areas. This is indicative of the higher total coliform concentrations normally found in CSO's (5 to 10 x $10^6$ MPN per 100 ml) as compared to storm drainage (2 x $10^6$ MPN per 100 ml). However, to achieve this increased control level, an additional expenditure of about $5.3 million (present worth) would be required, primarily for the new storm drains. This would result in a total estimated 1980 present worth cost of $9.80 million for Alternative D-J.

Comparison of Commercial Point Storage/Containment Alternatives. In addition to Table XI-1 which contains pertinent information associated with the alternatives, Figure XI-9 was developed to illustrate the relationship between estimated costs and CSO control levels. As shown, the estimated cost to achieve a control level of about 3 violation days per year would be about the same for each of the alternatives except D-I and D-J which, at most, can achieve control levels of 16 and 6 days per year, respectively. For control levels greater than 3 (i.e., 1 and 2 violation days per year), it would be more cost-effective to route the drainage from the separate sewered areas directly to the bay than to provide additional off-line storage for this runoff. For control levels less than 3 (i.e., 4 to 12 violation days per year), the opposite is true - that is it would be more cost-effective to store the drainage from the separate sewered areas than to route it directly to the bay. Figure XI-9 indicates that the more cost-effective alternative between the 3 and 12 violation days per year control levels is D-F, with D-J being cost-effective within the 6 to 7 days per year control level. However, considering that the present worth costs of Alternates D-E and D-F are within 10 percent of each other, both alternates can be considered equal. The D-F curve appears to "break" at about the 4 day per year control level. Based on the estimated 1980 present worth cost for Alternative D-J of $9.80 million, the unit cost to reduce CSO violation day frequency from 24 (assuming the Hoyt Street regulator is modified) to 6 days per year would be about $0.54 million per violation day reduction. For Alternative D-F, (or D-E) the unit cost to achieve a 4 days per year control level would be about $0.60 million per violation day reduction (1980 present worth cost of $12.14 million). Since these unit costs are relatively close, and considering Alternative D-F (and D-E) serves both the combined plus separate sewered areas resulting in a 40 percent larger tributary area, Alternatives D-Fand D-E are the most cost-effective storage/containment alternatives for the Commercial Point tributary area. The facilities associated with Alternative D-F are shown on Figure XI-10. The 1980 present worth cost is estimated to be $12.14 million.

Dorchester-Entire Area (Alternative D-L). This alternative was developed primarily to determine if it is more cost-effective to provide CSO control for the entire Dorchester area (i.e., areas tributary to BOS-088/089 and BOS-090) at a single location. To implement this alternative, CSO's would be routed from the Commercial Point tributary area (BOS-090) to the BOS-089 outlet serving the Fox Point tributary area. As shown on Figure XI-6, new CSO conduits would be required, located generally parallel to existing conduits, to provide increased conveyance capacity from the

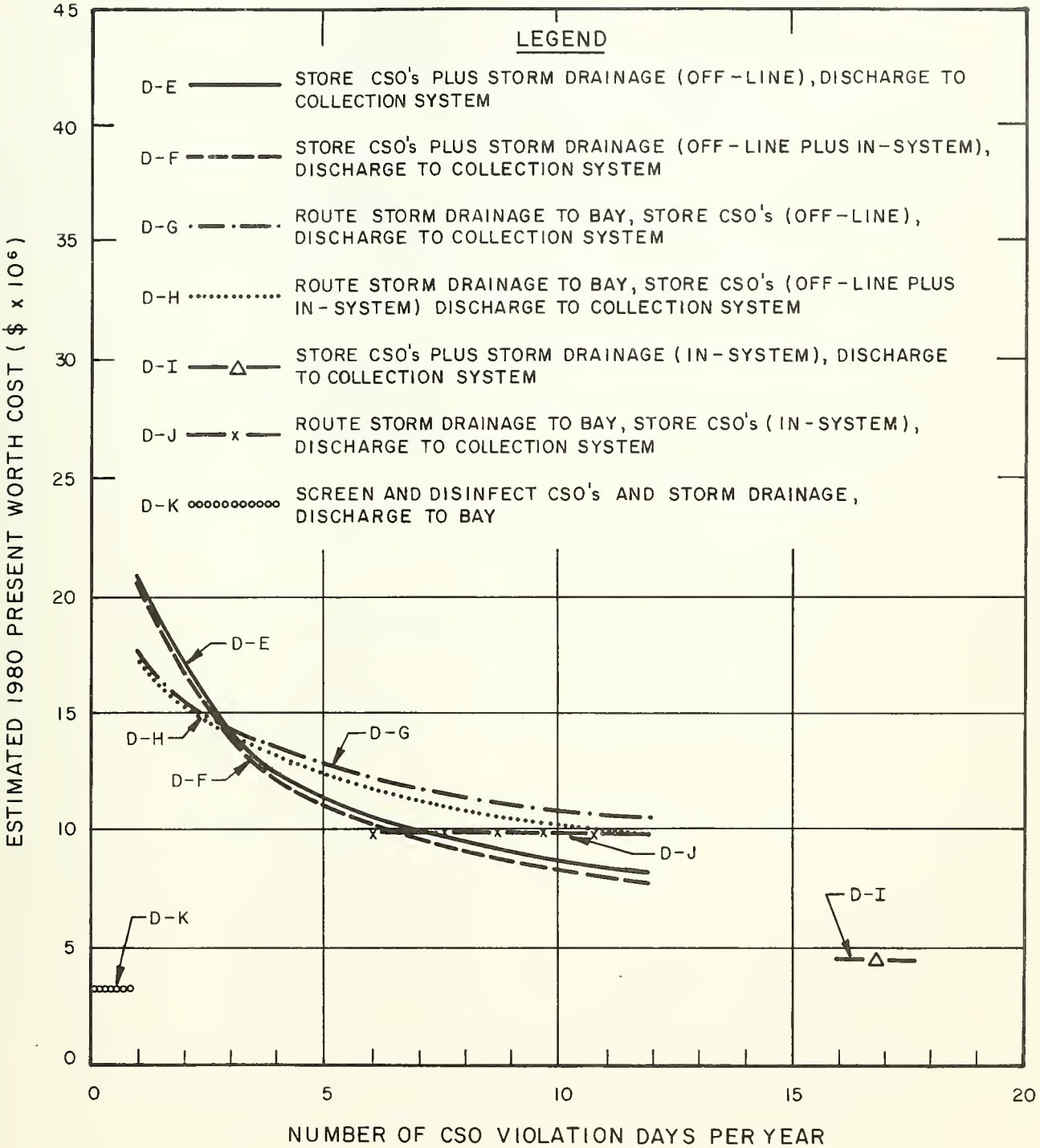
LEGEND
D-E STORE CSO's PLUS STORM DRAINAGE (OFF-LINE), DISCHARGE TO COLLECTION SYSTEM
D-F STORE CSO's PLUS STORM DRAINAGE (OFF-LINE PLUS IN-SYSTEM), DISCHARGE TO COLLECTION SYSTEM
D-G ROUTE STORM DRAINAGE TO BAY, STORE CSO's (OFF-LINE), DISCHARGE TO COLLECTION SYSTEM
D-H ROUTE STORM DRAINAGE TO BAY, STORE CSO's (OFF-LINE PLUS IN-SYSTEM) DISCHARGE TO COLLECTION SYSTEM
D-I STORE CSO's PLUS STORM DRAINAGE (IN-SYSTEM), DISCHARGE TO COLLECTION SYSTEM
D-J ROUTE STORM DRAINAGE TO BAY, STORE CSO's (IN-SYSTEM), DISCHARGE TO COLLECTION SYSTEM
D-K SCREEN AND DISINFECT CSO's AND STORM DRAINAGE, DISCHARGE TO BAY
ESTIMATED 1980 PRESENT WORTH COST ($ x 10⁶)
NUMBER OF CSO VIOLATION DAYS PER YEAR
0
5
10
15
20
25
30
35
40
45

FIG. XI-9 EVALUATION OF SELECTED ALTERNATIVES FOR DORCHESTER AREA TRIBUTARY TO BOS-090 (COMMERCIAL POINT)

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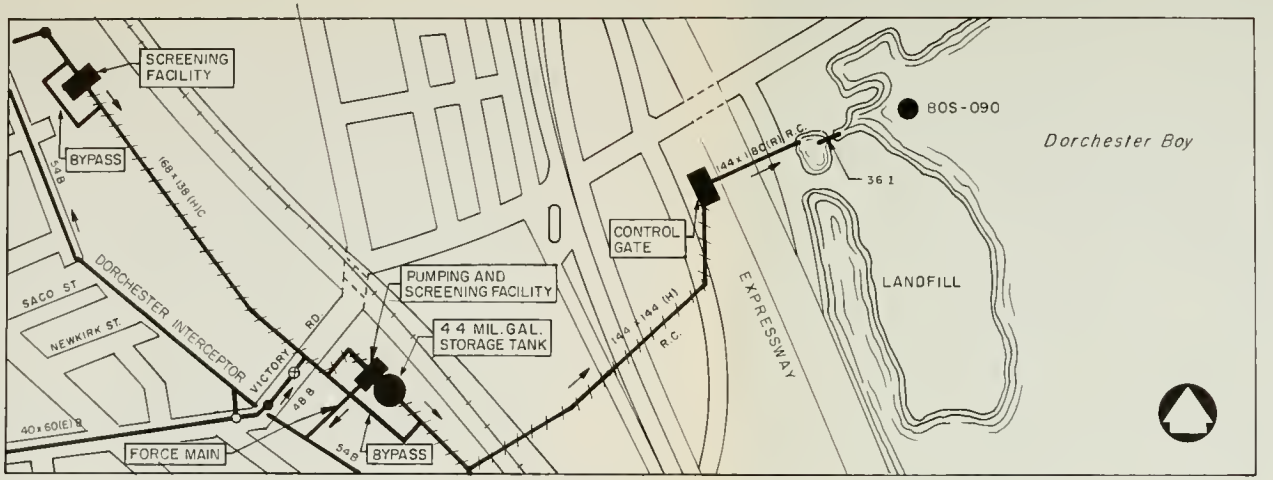



PLAN

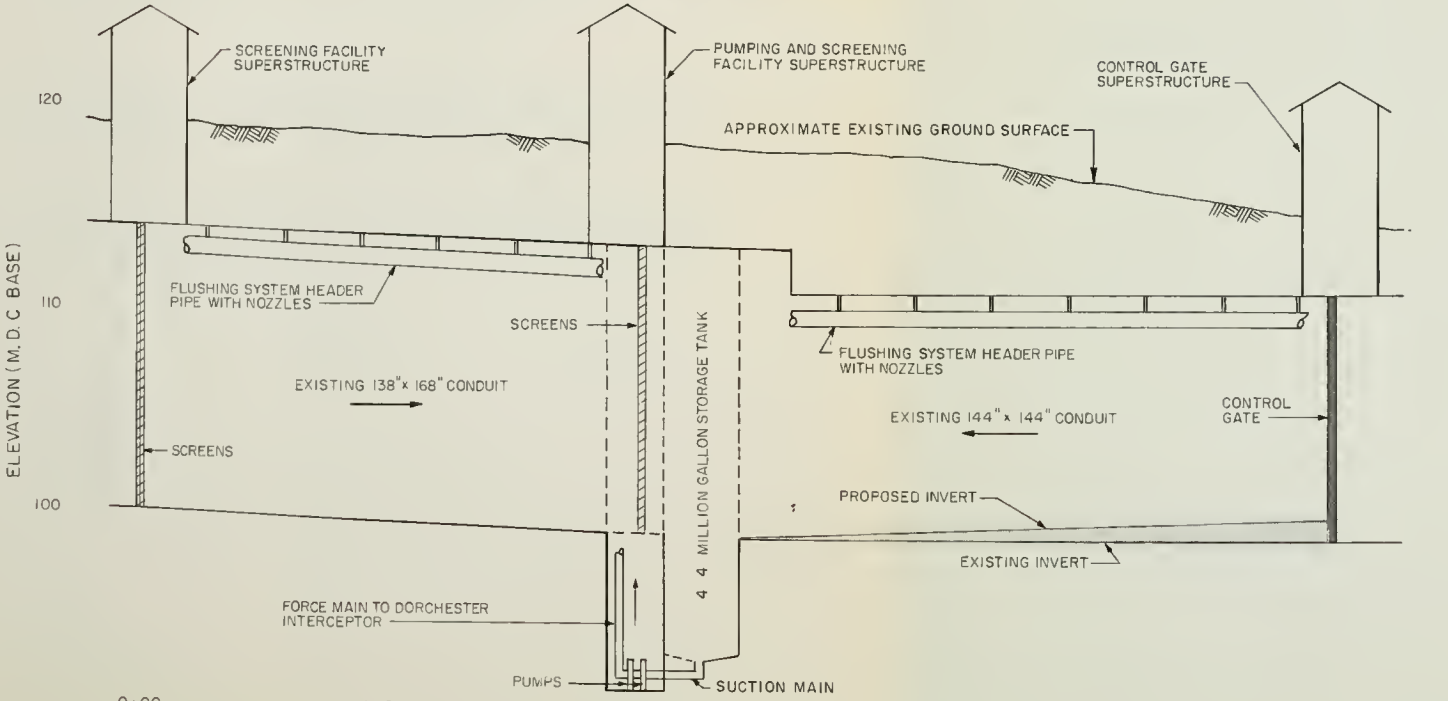
400 0 400 800

SCALE IN FEET

LEGEND

CONDUITS USED FOR IN-SYSTEM STORAGE

SUPERSTRUCTURE




SECTION

SCALE IN FEET

0 5 10 VERTICAL

100 0 100 200 300 400 HORIZONTAL

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COMBINED SEWER OVERFLOW PROJECT

DORCHESTER BAY AREA FACILITIES PLAN

COMMERCIAL POINT

STORAGE/CONTAINMENT ALTERNATIVE (D-F)

FIG. XI-10

existing BOS-090 regulator overflows to the BOS-089 CSO conduit. The conveyance capacity provided (i.e., the size of the new conduits) would depend upon the control level selected. Modifications to six regulators within the BOS-090 tributary area would also be required, in order to direct this additional CSO volume to the new conduits. In the event the capacity of the new conduits is exceeded, overflows would occur at the modified regulators and discharge to existing BOS-090 CSO conduits for discharge to the bay at the BOS-090 outlet at Commercial Point. Runoff from the 340 acres of separate sewered area within the Commercial Point tributary area would continue to discharge directly to the bay via BOS-090. The size of the new CSO conduits would range from 36-in. diameter to 72-in. diameter and were preliminarily sized based on actual storm events by the method described earlier in this chapter.

Under this alternative, off-line storage facilities would be provided at the Malibu Beach parking lot site. The required size of the storage tank for each control level was based on storing the CSO's from the Fox Point and Commercial Point tributary areas. It was also assumed that the Savin Hill storm drainage would be routed to the bay for the reasons discussed earlier. Required off-line tank sizes would be as follows:

| Control Level (Violation Days per Year) | Required Storage Volume (mil. gal.) |
|---|---|
| 1 | 11.6 |
| 4 | 5.3 |
| 12 | 1.9 |

The storage facilities would be the same as those described for Fox Point Alternative D-A, and would include a superstructure containing screens, dewatering pumps, hoppers, odor control equipment and controls.

The estimated costs for each control level under this alternative are given in Table XI-1. Figure XI-11 shows a plot of estimated 1980 present worth costs vs. the number of CSO violation days per year. A comparison between this alternative and the most cost-effective alternatives for each separate tributary area is as follows:

| Comparison Criteria | Alternatives D-C Plus D-F | Alternative D-L |
|---|---|---|
| Average Control-level-for Comparison Purposes (violation days/yr) | 6.5 | 6.5 |
| Total 1980 Present Worth Costs (million $) | 17.46 | 15.00 |
| Tributary Area Served (acres) | 1,266 | 926 |
| 1980 Unit Present Worth Cost per Acre | $13,800 | $16,200 |
| Estimated Annual Total Coliform Load to Bay (pseudo-lbs x $10^8$) | 53 | 79 |

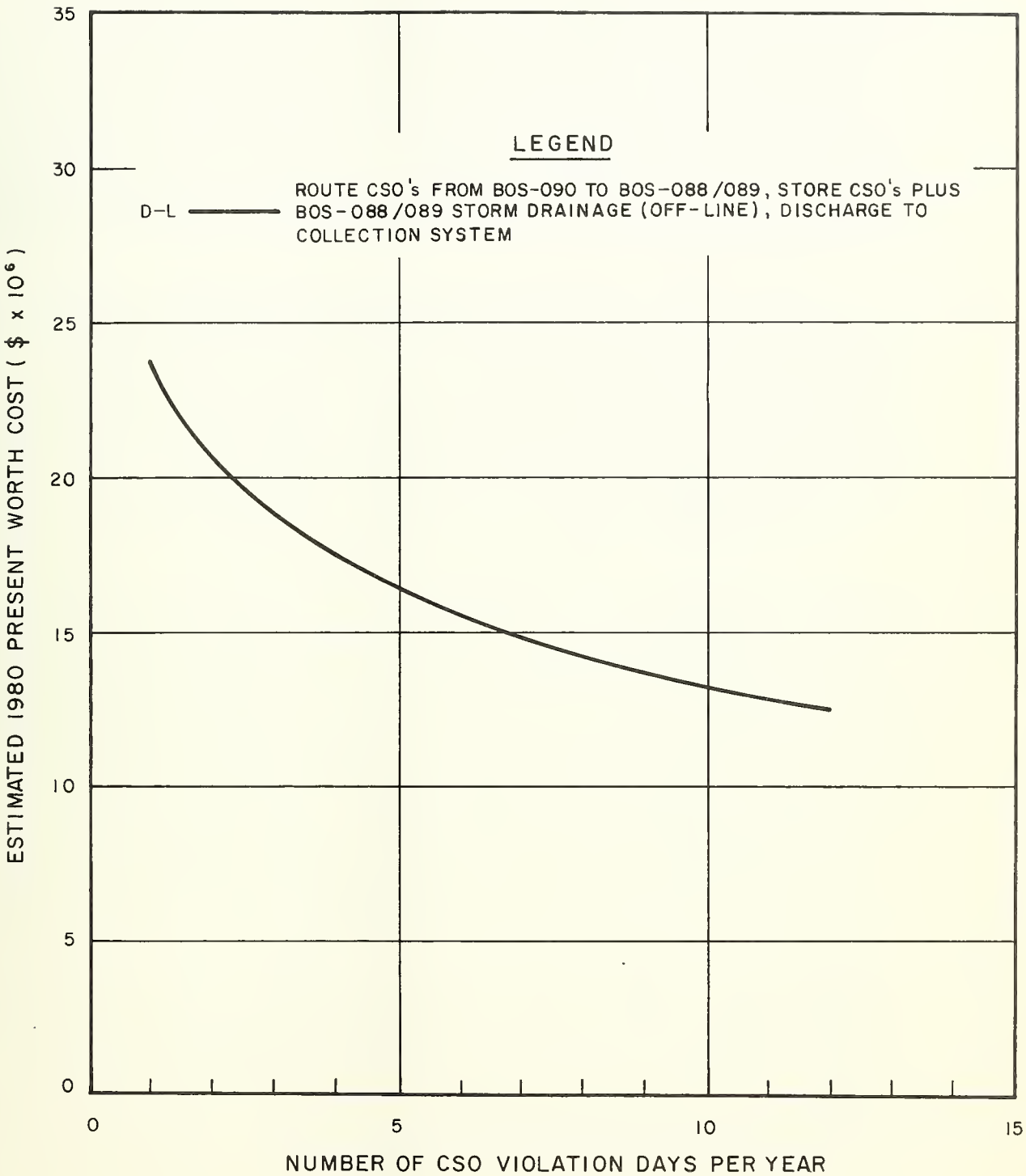


FIG. XI-11 EVALUATION OF SELECTED ALTERNATIVES FOR ENTIRE DORCHESTER AREA (TRIBUTARY TO BOS-088/089 AND BOS-090)

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COMBINED SEWER OVERFLOW PROJECT
DORCHESTER BAY AREA FACILITIES PLAN

Alternatives D-C plus D-F have a lower 1980 present worth unit cost per acre than Alternative D-L, and the total annual coliform load to the bay would be reduced by 26 x $10^8$ pseudo-lbs (33 percent). An additional advantage associated with Alternatives D-C plus D-F is the relatively minor adverse environmental impacts during and after construction as compared to Alternative D-L which would result in major disruptions to heavily used roadways and possible major utility relocations during the construction of the required new CSO conduits and regulator modifications. For these reasons, it is concluded that the additonal expenditure for Alternatives D-C and D-F is warranted, accordingly these alternatives have been selected as the storage/containment alternatives for the Dorchester portion of the study area.

Structural Alternatives - Screening/Disinfection

As discussed in Chapter X, the unit process of screening with disinfection would satisfy the major study area objectives. The major intent of CSO disinfection is to reduce the short-term coliform impacts resulting from wet weather discharges on the study area's beaches. Disinfection would serve to reduce the likelihood of pathogenic organism survival, and the possibility of bathers contracting diseases of the gastrointestinal tract such as typhoid fever, dysentry and diarrhea. Accordingly, the concept of disinfection to be employed only during the bathing season (i.e., June through September) seems warranted.

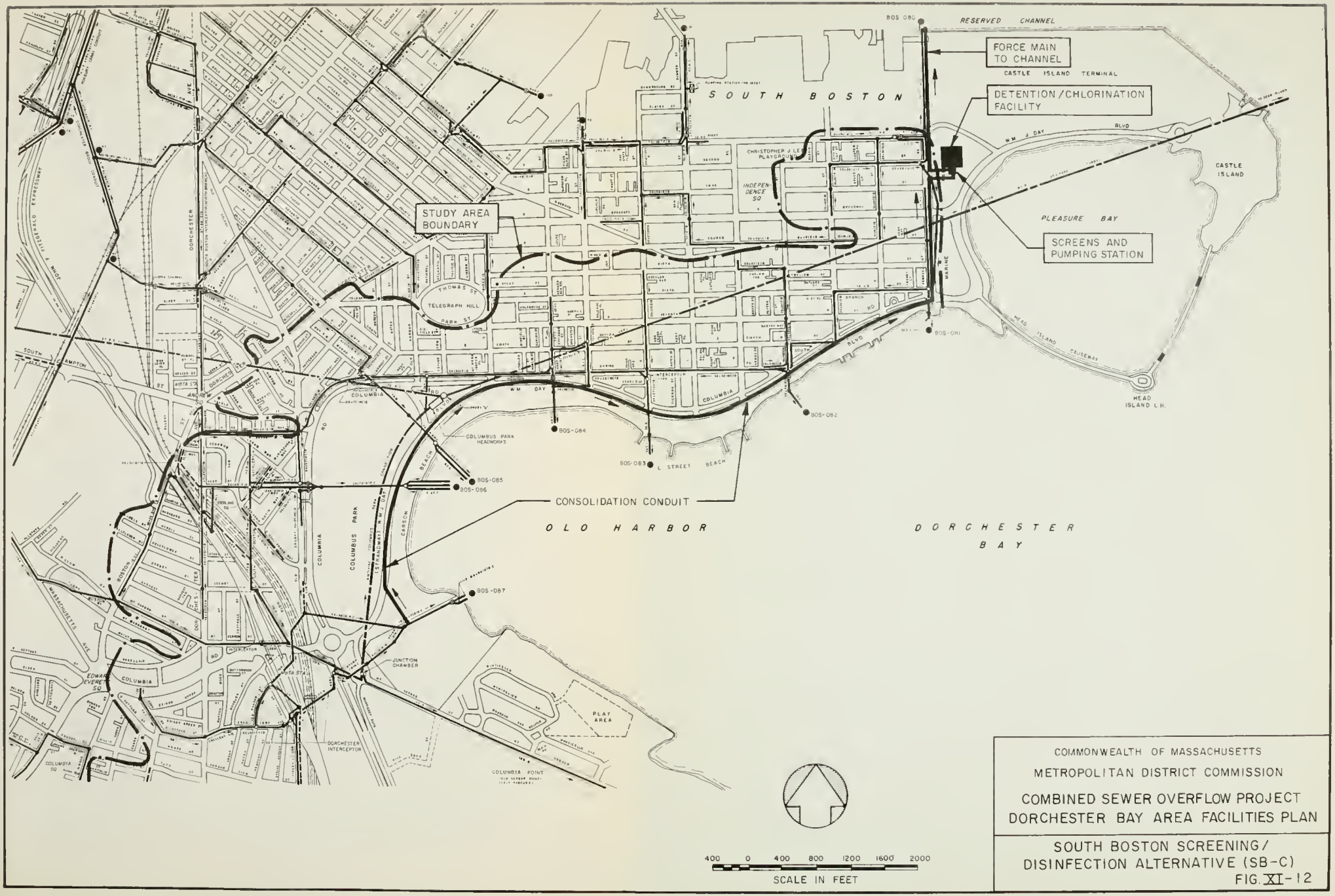
Several disinfection agents were examined during the detailed evaluation. These include chlorine, sodium hypochlorite, chlorine dioxide, bromine chloride, ozone, and ultraviolet (UV) radiation. The results of this evaluation indicated that disinfection utilizing sodium hypochlorite is the least costly and most widely used method. It also offers a relatively high degree of safety, since it is stored usually as a 12-15 percent liquid solution under atmospheric pressure. This is contrary to chlorine gas which is highly toxic and, upon contact, is dangerous to health since it can cause severe burns, irritations to the respiratory system and, under long exposure, death by suffocation. Safety is an important consideration considering the densely populated, highly urbanized study area. Accordingly, sodium hypochlorite has been selected as the disinfecting agent for the screening/disinfection alternatives.

In Chapter X, several concerns were raised regarding possible adverse effects to the marine environment due to the discharge of a chlorinated waste stream to the near shore waters of the bay. As stated in Chapter X, the EPA has recommended ("Quality Criteria for Water," July 1976) a maximum chlorine residual of 0.01 mg/l in the water column for protection of marine life. The persistance of chlorine residual, while necessary to ensure complete kill of pathogens and prevent aftergrowth, is the major cause of the toxic effects of chlorine on marine organisms. Since the chlorinated waste streams from study area screening/ disinfection facilities would discharge to Dorchester Bay, the size of the receiving water body as well as its typical characteristics must be evaluated.

Dilution ratios anticipated, and also considering the effects of wind, dispersion and tidal exchange as discussed in Chapter IX, the chlorine concentrations in the bay would, most likely, be negligible. Waters in the immediate vicinity of the outfalls may also have residual levels that meet the EPA recommended limit of 0.01 mg/l, if dosage rates can be adequately controlled. However, if higher residual concentrations were to occasionally occur near the outlets, it should not result in significant, long-term adverse effects to the marine ecosystem since disinfection would not occur on a continuous basis. Disinfection would only be used during the bathing season (approximately 4 months per year), and for those rainfall events that result in a discharge through CSO outfalls. Based on SEMSTORM results, the estimated average number of discharge events occurring during the period between June and September would be in the order of 20 to 25 for South Boston and Dorchester. It should be noted that for "smaller" storms, discharge would consist of stormwater runoff only but for "larger" storms the discharge would be combined stormwater and sanitary wastewater (i.e., CSO's).

An additional discussion regarding the impacts of these chlorinated discharges to the marine ecosystem is presented in Volume II-Chapters IV and V. A discussion of the screening/disinfection alternatives for each of the three major tributary areas follows.

South Boston. The South Boston screening/disinfection alternative is designated as SB-C. The major facilities associated with this alternative include a consolidation conduit serving areas tributary to CSO outlets BOS-080 through BOS-087, and a screening and detention/chlorination facility. As shown on Figure XI-12, the consolidation conduit would traverse the same route as described previously for Alternative SB-A, however flow would be conveyed to a treatment facility located in Marine Park. The size of the conduit would range from 48-in. diameter to 84-in. diameter for the 1 and 4 violation days per year control levels, and would be approximately 10,000 ft. long. It would generally be located in Day Boulevard, from BOS-087 outlet to BOS-081 outlet, and then extend north in Farragut Road to the treatment facility. The facilities' location in Marine Park was selected under this alternative, in order to allow the chlorinated waste stream to be discharged into Reserved Channel, which is currently used primarily for shipping purposes. This would eliminate discharges, and any associated adverse impacts, to the marine ecosystem of the Old Harbor portion of Dorchester Bay, which contains several beaches, shellfish areas and yacht clubs. Hydraulic relief of the consolidation conduit during those events when the capacity is exceeded (i.e., between 1 and 12 days per year depending on the control level) would occur through the existing CSO outlets. Stationary, manually cleaned bar racks would be installed on these outlets to prevent the discharges of large floatables during those relatively infrequent storm events.

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COMBINED SEWER OVERFLOW PROJECT
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SOUTH BOSTON SCREENING/
DISINFECTION ALTERNATIVE (SB-C)
FIG. XI-12

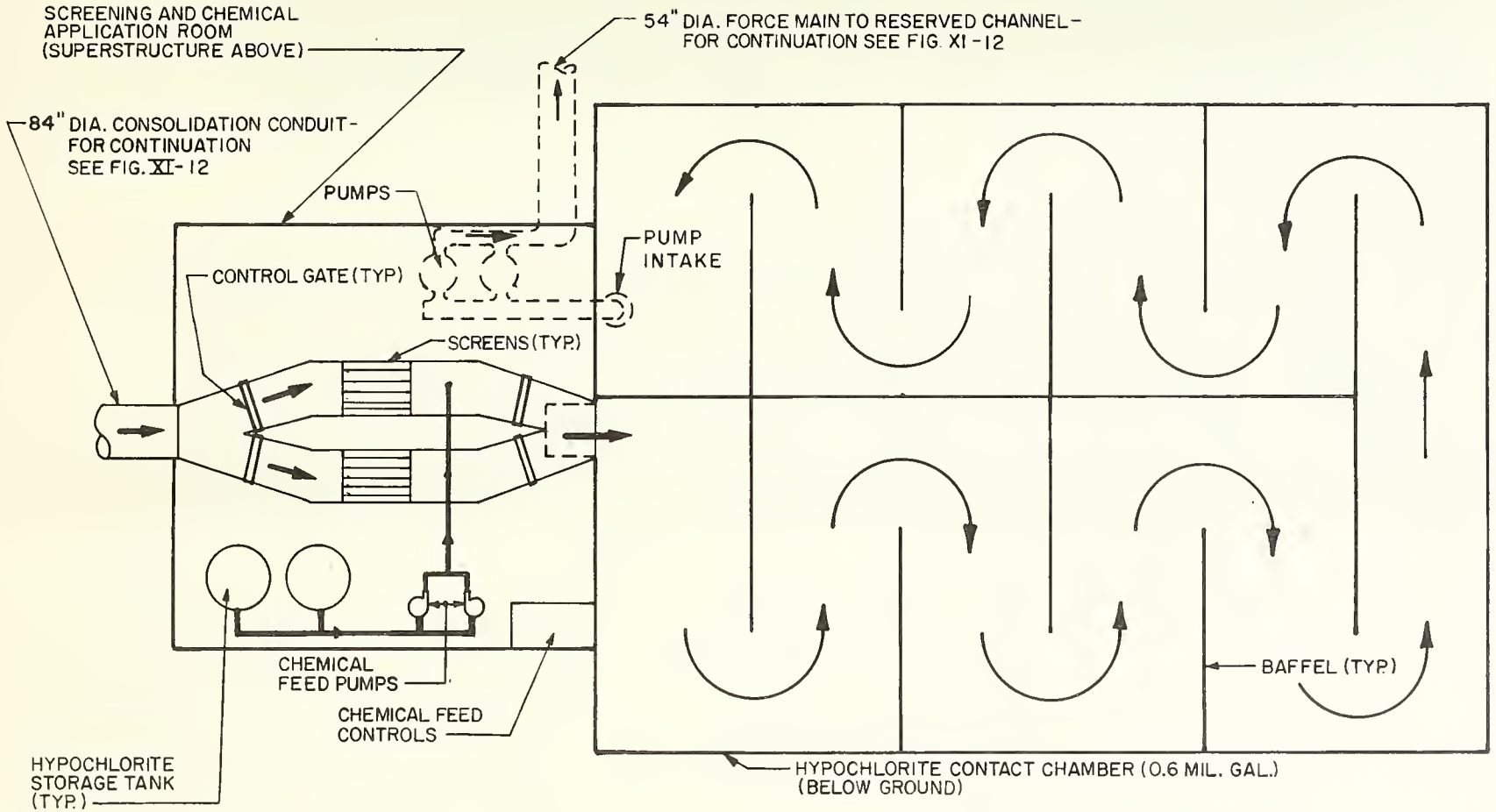
Details of the detention/chlorination facility are shown on Figure XI-13 for the 4 violation days per year control level. Flow would enter the screening chamber from the consolidation conduit and pass through coarse (1-1/2" bar spacing) and fine (3/4" bar spacing) mechanically cleaned screens. Sodium hypochlorite would be applied downstream of the screens with the dosage rate controlled by the rate of flow. The chlorinated waste stream would then enter the contact chamber designed to provide 10 minutes detention time during peak flow conditions. Baffles installed in the chamber would assure adequate mixing and efficient kill. The chlorinated waste stream would be pumped to the Reserved Channel for ultimate disposal.

As previously mentioned, the disinfectant would be applied only during those discharge events that occur during the bathing season (i.e., 4 months per year). Assuming a chlorine dosage level of 15 mg/l, it is estimated that about 7,100 gallons of 12 percent sodium hypochlorite solution would be required each year.

The estimated costs of this alternative are presented in Table XI-1 for each control level. Figure XI-5 illustrates the relationship between costs and CSO control levels for this alternative as well as the storage/containment alternatives. It is evident that the costs associated with this alternative are extremely close to the costs for Alternative SB-B, which is the most cost-effective storage/containment alternative.

Dorchester - Fox Point and Commercial Point. The screening/disinfection alternatives for the two Dorchester tributary areas are designated as D-D for the Fox Point tributary area, and D-K for the Commercial Point area. The facilities would be located on the two major CSO outlets serving the Dorchester area, as shown on Figure XI-14. Each would include automatically cleaned, coarse (1-1/2" bar spacings) and fine (3/4" bar spacings) screens arranged in series in the CSO conduit. The operation of the scraping mechanism would be controlled based on the head differential across the screens. The screenings would be discharged into collection hoppers, from where they would be emptied into a truck for ultimate disposal, most likely at the existing landfill located at Deer Island. Each facility would also include sodium hypochlorite storage and feed equipment, required for disinfection of CSO's during the summer months. For cost estimating purposes, and to estimate the required on-site hypochlorite storage volume, a dosage of 15 mg/l chlorine was assumed. This dosage may be somewhat conservative considering the existing MDC operated Cottage Farm Detention/Chlorination facility located on the Charles River currently applies hypochlorite at a rate equivalent to a 5 mg/l chlorine dosage. For this analysis however, it was assumed that the ability to apply chlorine at a rate up to 15 mg/l should be provided to assure adequate coliform kill under all conditions. The actual dosage would depend on pilot studies during the final design of the facility.

At each facility, detention time would be provided in the CSO conduit downstream of the facility. It is estimated that for the 1-year, 6-hour storm, detention times of about 17 minutes and 33 minutes would be available in the Commercial Point (BOS-090) and Fox Point (BOS-089) CSO conduits, respectively, under average flow conditions. This ability to


SCREENING AND CHEMICAL APPLICATION ROOM (SUPERSTRUCTURE ABOVE)
54" DIA. FORCE MAIN TO RESERVED CHANNEL - FOR CONTINUATION SEE FIG. XI-12
84" DIA. CONSOLIDATION CONDUIT - FOR CONTINUATION SEE FIG. XI-12
PUMPS
PUMP INTAKE
CONTROL GATE (TYP)
SCREENS (TYP.)
CHEMICAL FEED PUMPS
CHEMICAL FEED CONTROLS
HYPOCHLORITE STORAGE TANK (TYP.)
BAFFEL (TYP.)
HYPOCHLORITE CONTACT CHAMBER (0.6 MIL. GAL.) (BELOW GROUND)
20 0 20 40
SCALE IN FEET

PLAN

NOTE: FOR A LOCATION PLAN OF THIS FACILITY REFER TO FIG. XI-12



FIG. XI-13 SOUTH BOSTON DETENTION/CHLORINATION FACILITY (ALT. SB-C)

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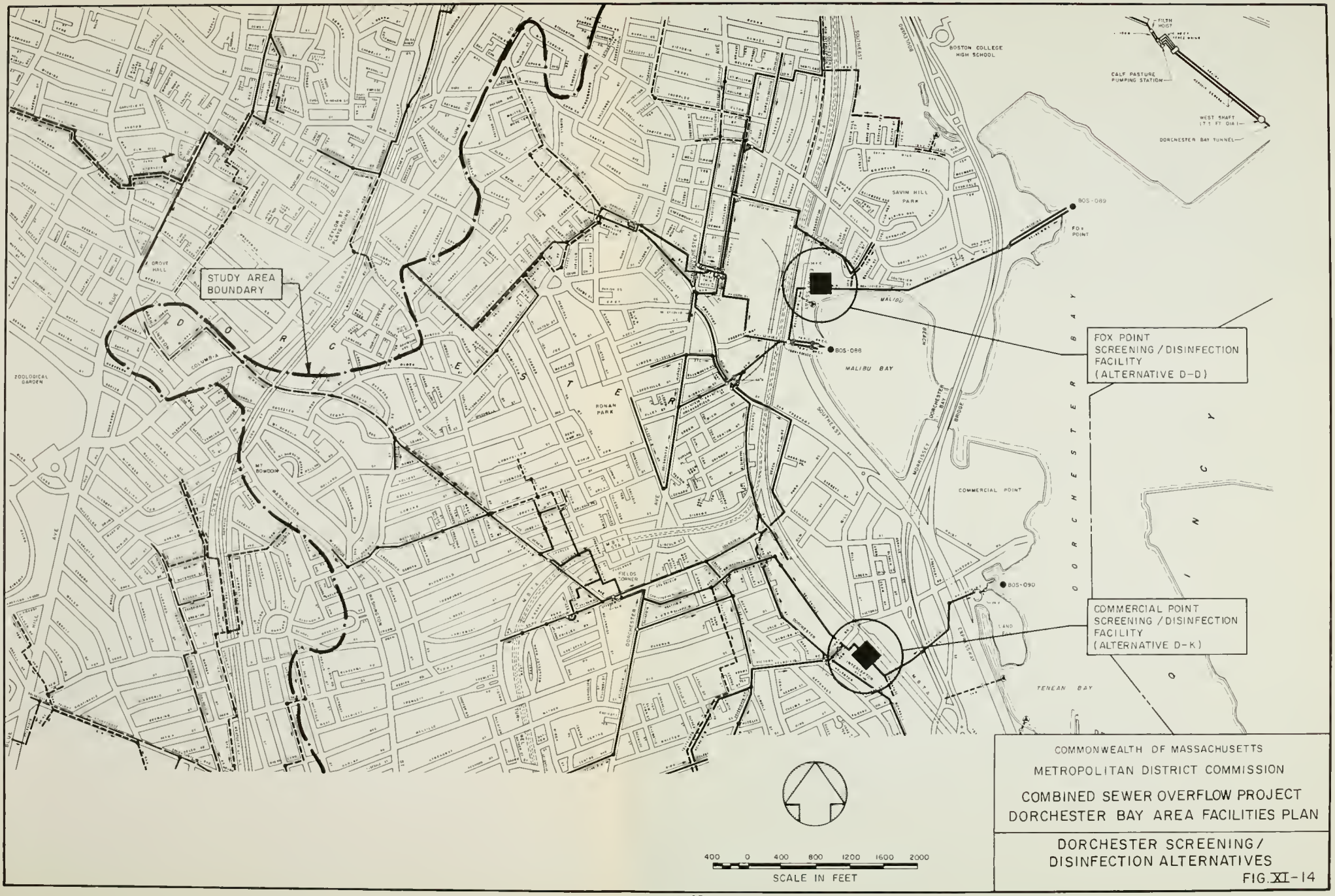


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COMBINED SEWER OVERFLOW PROJECT

DORCHESTER BAY AREA FACILITIES PLAN

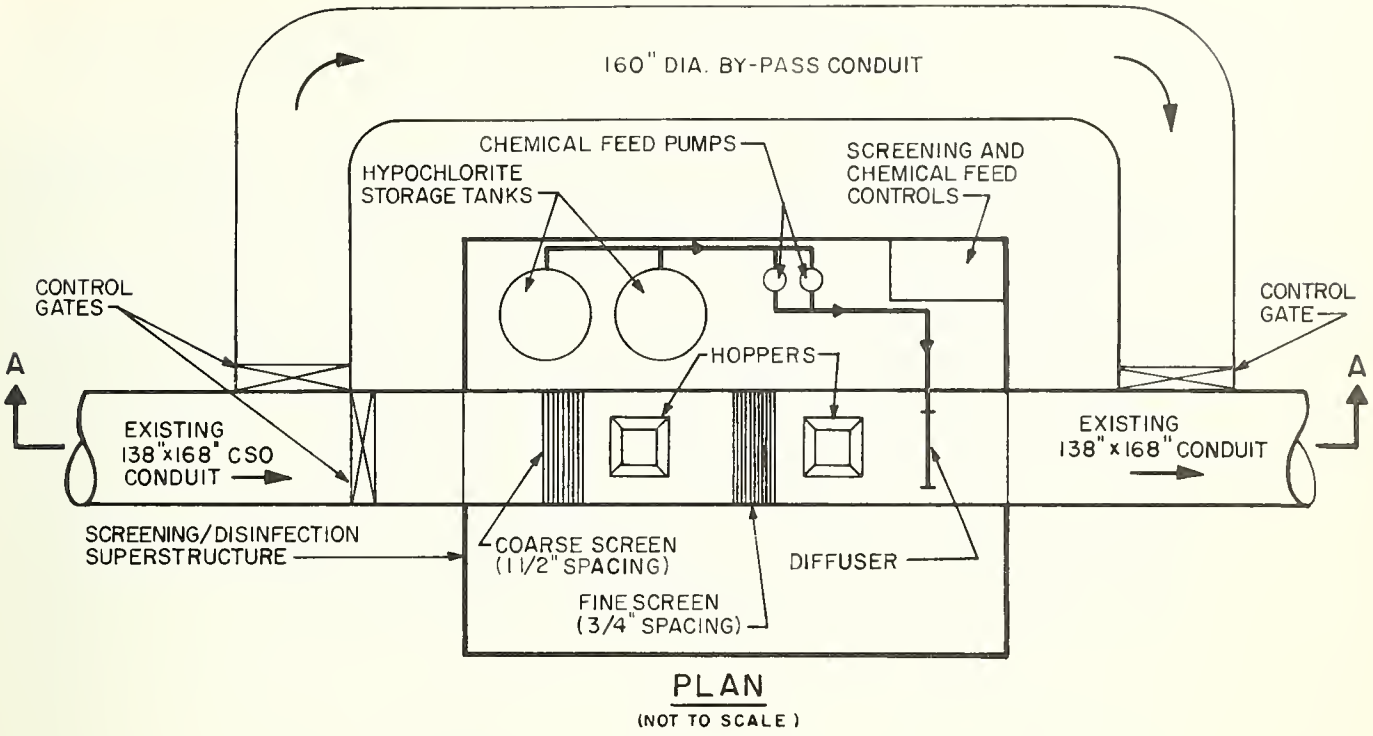
DORCHESTER SCREENING / DISINFECTION ALTERNATIVES

FIG. XI-14

utilize existing conduits for detention time eliminates the need for a chlorine contact chamber, and is the primary reason for the cost-effectiveness of this alternative in the Dorchester area. The installation of baffles in the conduits may be required to assure adequate mixing, however, this will be determined during final design and pilot testing. Assuming disinfection would occur between June and September, it is estimated that approximately 2,000 gallons of 12 percent solution sodium hypochlorite would be required per year at the Fox Point facility, and 6,000 gallons per year at the Commercial Point facility. Hypochlorite storage facilities equivalent to these volumes would be provided.

A bypass conduit would be required at each facility for emergency relief in the event of screening equipment failure, and also would be required during construction of the facilities in order to provide continuous operation of the CSO conveyance system. A bypass conduit would also allow the option of the facility being constructed on the bypass conduit, with the existing CSO conduit being utilized for emergency bypass purposes. This would provide additional flexibility for locating the superstructure (approximately 50' x 30' x 12'-15' high) at a site offering the least disruption to normal site activities. At McMorrow Playground (Commercial Point area), the facility could be located away from existing recreational fields in a corner of the field, resulting in a minimal disruption to normal recreational activities. For the Fox Point area, the facility could be located on a state-owned vacant lot located adjacent to the Dorchester Yacht Club access road, and would result in minimal, if any disruptions to normal area activities. Final site selection will be based on field surveys performed during the preliminary design phase of this project (Chapter XII).

A preliminary layout of the Commercial Point facility is shown on Figure XI-15. The Fox Point facility would essentially be the same. Estimated costs for each alternative (D-D for Fox Point and D-K for Commerical Point) are given in Table XI-1. The estimated 1980 present worth cost of these alternatives is $3.88 million for D-D, and $3.08 million for D-K, for a total of $6.96 million. Their cost-effectivenss is apparent, if compared to the storage/containment alternatives shown on Figures XI-8 and XI-9. Since each facility would have the capability of screening and disinfecting flows close to the capacity of the existing system which was designed to convey 15-year frequency storm flows, the potential to achieve a control level equal to or greater than 1 CSO violation day per year exists. Floatables removal would be provided to this control level, however disinfection is warranted only during the bathing season for the reasons presented earlier. Volume II, Chapters IV and V discuss the effect of these alternatives on the physical, social and biological environments.




PLAN

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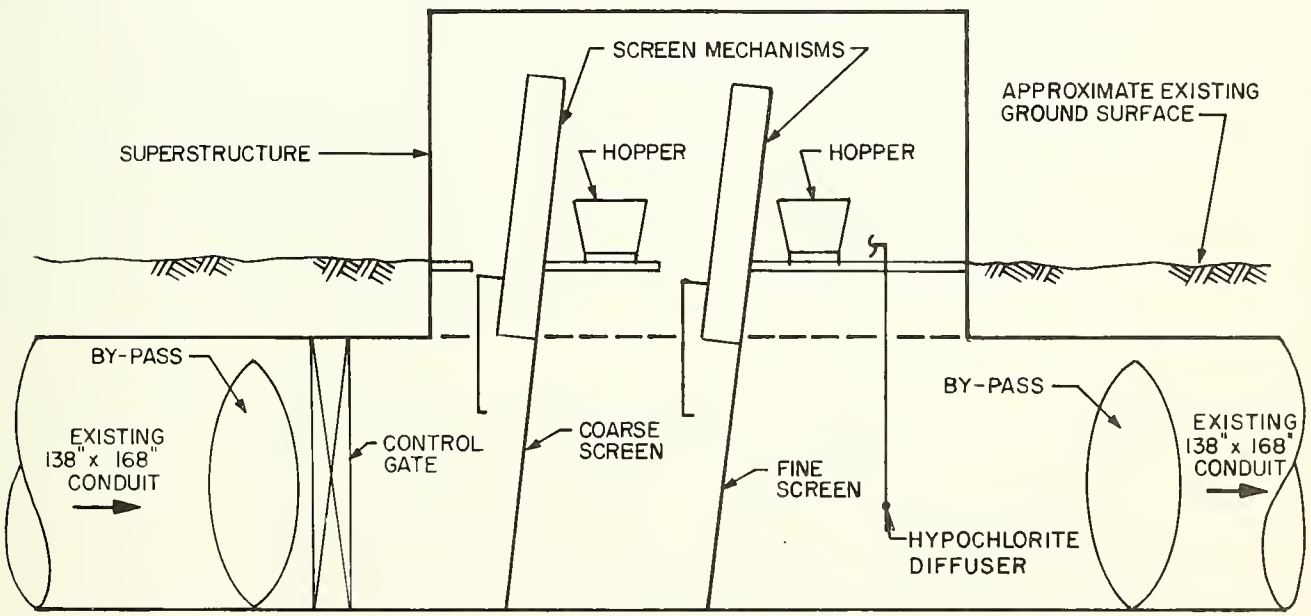


SECTION A-A

(NOT TO SCALE)

NOTE:
FOR A LOCATION PLAN OF THIS FACILITY
REFER TO FIG. XI-14

FIG. XI-15 COMMERCIAL POINT SCREENING/ DISINFECTION FACILITY (ALTERNATIVE D-K)

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COMBINED SEWER OVERFLOW PROJECT
DORCHESTER BAY AREA FACILITIES PLAN

South Boston. As previously discussed and shown on Figure XI-5, the estimated costs for the three South Boston structural alternatives are, for all practical purposes, the same for the control level range examined (i.e., 1 to 12 CSO violation days per year). This is primarily due to the construction cost of the consolidation conduit, which is required under all alternatives. In addition, all alternatives involve the construction of an underground tank and an adjacent superstructure containing screens, pumps, controls and other related equipment. Although the Alternative SB-C (screening/disinfection) tank is smaller, because its size was based on providing a disinfectant contact time only, the present worth costs of this alternative are comparable to the storage/containment alternatives because of the additional 2,000-ft. of consolidation conduit (in Farragut Road), a force main from the disinfection facility to the Reserved Channel, and hypochlorite storage and feed equipment. Accordingly, the selection of the South Boston recommended alternative must be based on other considerations than estimated present worth costs only.

Alternative SB-C would result in the discharge of a screened and chlorinated waste stream to the Reserved Channel. However, for a comparable or possibly slightly lower present worth cost (Alternative SB-B within the 1 to 7 violation days per year range), the storage/containment alternates would substantially reduce wet weather waste streams to area waters, since CSO's would be stored and bled back to the collection system. Inherent removal of heavy metals, nutrients and other CSO related pollutants would also be obtained by storage/containment. This distinct advantage of storage/containment is important in the South Boston area, since about 87 percent of the study area's beaches (approximately 200 acres), 65 to 70 percent of its shellfish beds (approximately 200 acres), and four of its yacht clubs are impacted by wet weather discharges from this portion of the study area.

Alternative SB-B, which serves a larger area than SB-A and appears to be slightly more cost-effective, is selected as the recommended South Boston structural alternative. The slope of Alternative SB-B's cost-effectiveness plot (Figure XI-5) appears to "break" at the 4 violation days per year control level. This level appears to be a realistic goal since it would result in a statistical occurrence of about one violation day per bathing season. The additional expenditure of approximately $6 million (total present worth) to obtain a control level of one overflow per year is not considered warranted. Thus, Alternative SB-B, with facilities sized to result in a CSO control level of about 4 violation days per year, is selected. The facilities required under this alternative were described earlier and are shown on Figures XI-3 and XI-4. The preliminary sizes shown are reflective of the 4 days per year control level. The total estimated preliminary 1980 present worth cost is $17,800,000.

Dorchester. Unlike the South Boston alternatives, the cost effectiveness of the Dorchester screening/disinfection alternates (D-D for Fox Point and D-K for Commercial Point) is apparent. Also the number of CSO outlets and the uses of the receiving waters in the immediate location of the outlets are different in Dorchester. The Dorchester CSO outlets do

not discharge directly onto bathing beaches, as South Boston outlets do. Accordingly, the discharged CSO's in Dorchester have a tendency to dilute and disperse prior to reaching the bathing areas, although probably not enough to have no impact on the beaches.

Since Dorchester screening/disinfection facilities would have the capability of screening and disinfecting flows close to the capacity of the existing systems (designed to convey 15-year frequency storm flows), the potential exists to achieve a control level equal to or greater than 1 CSO violation day per year. Floatables removal would be provided to this control level (i.e., 1-year, 6-hour storm); however, disinfection appears warranted only during the summer bathing season since its primary purpose is to reduce the short-term coliform impacts resulting from CSO discharges on area beaches. This would reduce the liklihood of pathogenic organism survival and the possibility of bathers contracting diseases of the gastrointestinal tract. Disinfection of CSO's occurring during the non-bathing months would not be necessary, since it was demonstrated in Chapter X that the long-term SB coliform standards would, most likely, be met if dry weather overflows (DWO's) were removed and the Pine Neck Creek storm drain diverted away from Tenean Beach.

As discussed earlier in this chapter, it is estimated that discharge of this chlorinated waste stream during the summer months would not result in any major long-term adverse effects to the marine environment. This is based on the amount of dilution offered by the receiving waters, as well as the beneficial effects of wind, discharge dispersion, and the bay's tidal exchange which would serve to reduce the chlorine levels in the bay to negligible amounts. Since the two major Dorchester area CSO outlets (BOS-089 and BOS-090) are not located in the immediate vicinity of area beaches, the likelihood of objectionable levels of chlorine compounds such as chlorinated hydrocarbons occurring on these beaches is remote due to the above-mentioned receiving water characteristics. At each facility, the capability of residual chlorine level monitoring, at selected receiving water locations where chlorine levels would be of concern, would be provided. This, in addition to the monitoring of coliform kill efficiency, would allow the determination of the most effective chlorine dosage and would assure non-objectionable receiving water residuals.

The screening/disinfection alternatives in Dorchester would also result in considerably less adverse environmental impacts during and after construction, as compared to the storage/containment alternatives. The required facilities at each location would consist of a superstructure housing the screens, disinfection equipment and controls, and a bypass conduit around the facility. The flexibility of locating the facility at the least objectionable location on the site would be available by constructing it on the bypass conduit which could be routed as required. Thus, the disruption to normal site activities by the construction of a storage tank would be eliminated.

Based on the above discussion, the screening/disinfection alternatives D-D for the Fox Point tributary area, and D-K for the Commercial Point tributary are recommended. The required facilities have been described previously, with a typical installation shown on Figure XI-15. The total estimated 1980 present worth cost of the facilities are $3,880,000 for Fox Point and $3,080,000 for Commercial Point.

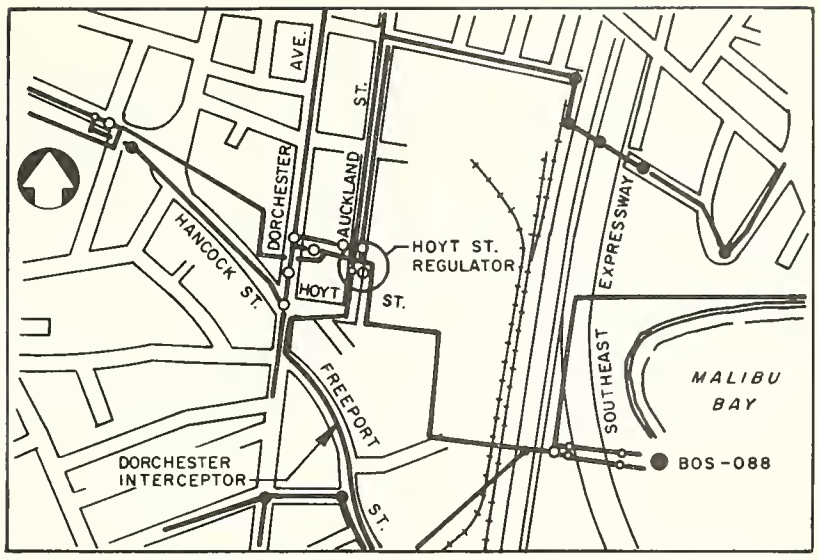
## Additional Recommendations

Hoyt Street Regulator Modifications. As discussed in Chapter X, it was found that by modifying the regulator located on the Dorchester interceptor near Hoyt Street, the interceptor could be operated up to a maximum surcharge capacity of about 60.0 mgd. Under the Hoyt Street regulator's current configuration, flows in excess of about 28.0 mgd (nominal interceptor capacity) result in an overflow at this location. By preventing this overflow occurrance and allowing the interceptor to flow at 60 mgd, the estimated annual number of CSO violation days would be reduced from an estimated current number of 53 to about 24 (refer to Figure X-7 in Chapter X).

Figure XI-16 illustrates the extent of modifications required to the Hoyt Street side overflow weir - type regulator. As shown, a control gate would be installed on the existing overflow channel in the adjacent tide gate manhole. A control gate is preferred over the permanent blocking of the regulator in order to allow the ability to hydraulically relieve the interceptor during extreme flow conditions. The gate would, most likely, be either a sluice gate or a slide gate equipped with a manual operator located in a vandal-proof enclosure at ground level. Considering the probable infrequent use of the gate, an automatically and power operated controlled gate was not considered to be warranted. Location of the gate in the tide gate manhole, in lieu of the overflow weir manhole, would facilitate installation and reduce costs considering the relatively dry conditions and ground accessibility of the manhole. In addition, it is also suggested that the invert of the overflow channel, which connects the weir to the tide gate chamber, be reshaped and sloped as shown in Figure XI-16 to aid in the prevention of solids deposition following a high flow condition in the interceptor (i.e., when the depth of flow exceeds elevation 103.00).

These modifications will be explored in detail during the preliminary design phase and are included as part of the total recommended plan. The total estimated preliminary capital cost of these modifications is $60,000 (June 1979 price levels - ENR 2900), and the 1980 present worth is estimated to be $65,000.

Pine Neck Creek Storm Drain Relocation. In Chapter X, the need was demonstrated for diverting the storm drainage emanating from the Pine Neck Creek storm drain away from Tenean Beach. With dry weather overflows removed, and considering that disinfection of Dorchester area CSO's would occur only during the summer months, relocation of the storm drain




LOCATION PLAN

400 0 400 800

SCALE IN FEET

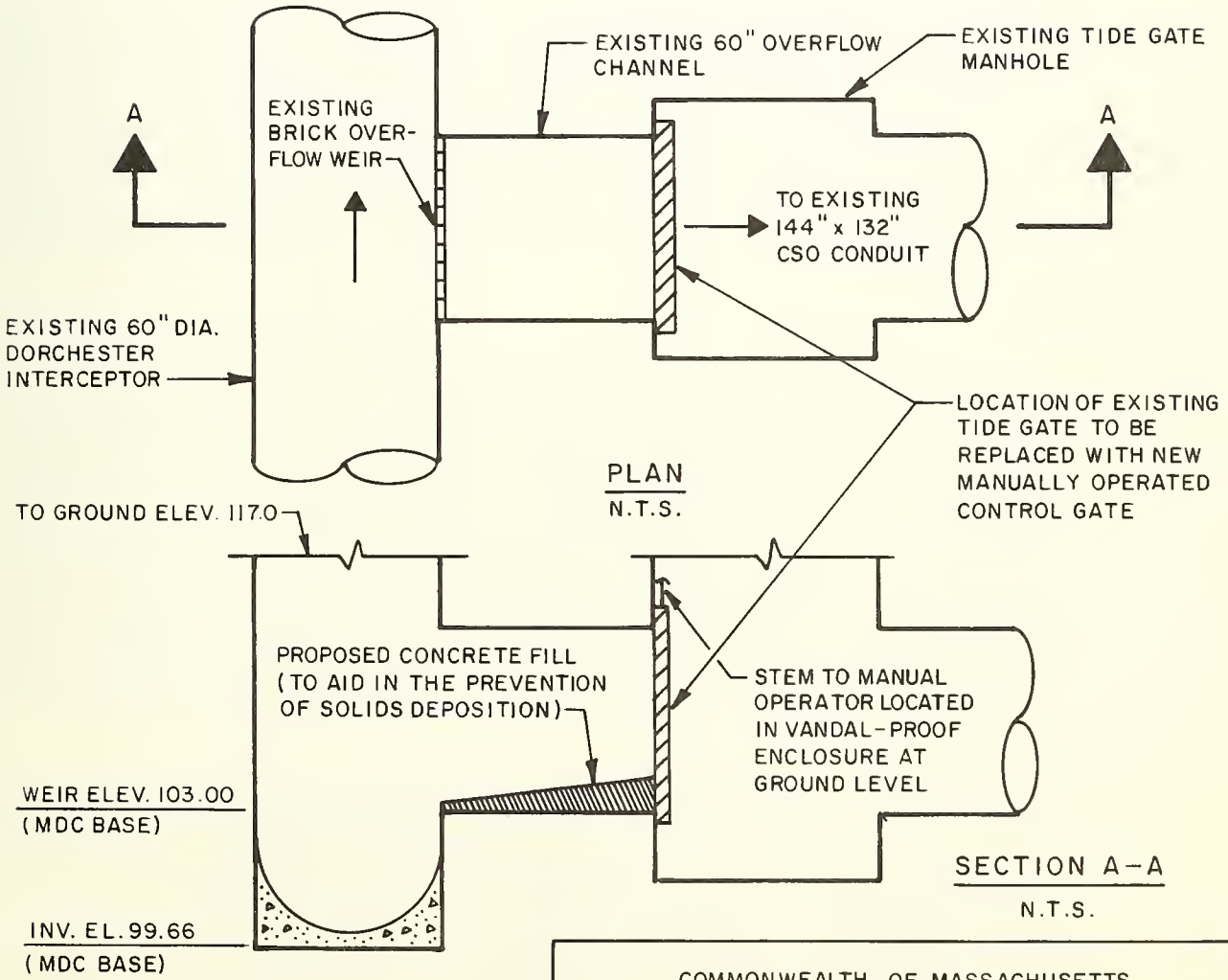



**FIG. XI-16 HOYT ST. REGULATOR MODIFICATIONS**

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is a measure that would serve to improve the likelyhood that the long-term water quality of Tenean Beach will continuously meet SB water quality standards. Currently, during low tide periods, Pine Neck Creek is the only source of water feeding Tenean Beach. Thus, during wet weather periods coinciding with low tides, the water quality approaches the quality of the stormwater runoff from the storm drain's 118 acre tributary area. Considering that this tributary area is highly urbanized, the runoff, in addition to containing an average total coliform concentration of about $2 \times 10^6$ MPN per 100 ml, would most likely also contain oil and grease, floatables and debris that may eventually deposit on the beach area. Thus, from an aesthetic and overall public health standpoint, the need to relocate the Pine Neck Creek storm drain is apparent.

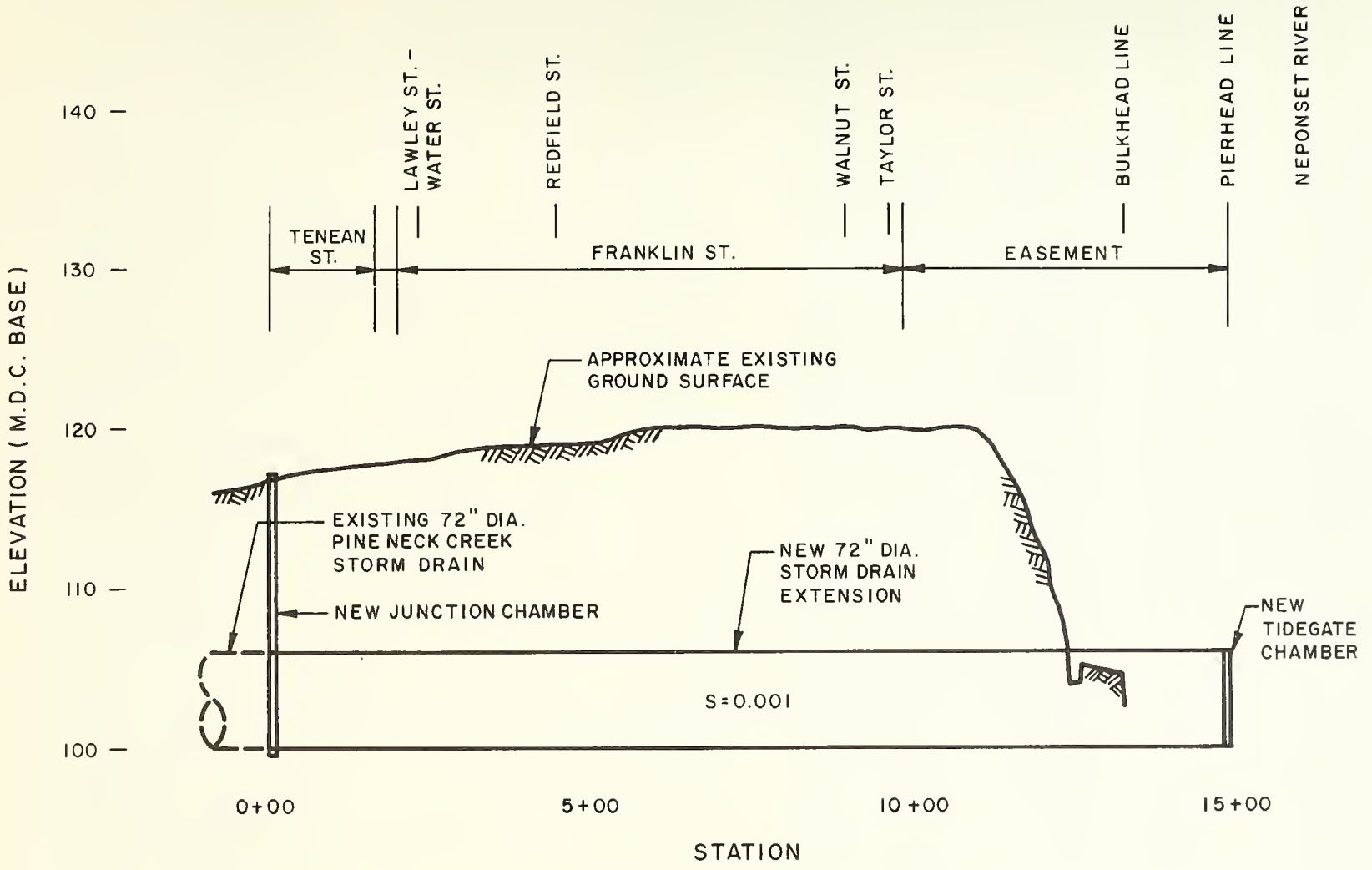
Three alternate routes for extending the existing 72-in. diameter storm drain to the Neponset River were preliminarily investigated during this phase. It appears that the most cost effective route, as well as the most feasible from a construction and easement acquisition standpoint, is the route shown on Figure X-11 in Chapter X. This route will be verified during the preliminary design phase and possibly revised based on field and subsurface investigations.

As conceived, the existing conduit would be intercepted by the new conduit in the vicinity of the tide gate chamber located in Tenean Street. The new drain would be 72-in. in diameter, and would run south in Tenean Street to Lawley Street; in Lawley Street to Franklin Street; and southeast in Franklin Street to privately owned land where it would continue in an easement to the Neponset River. The total length of the new drain along this route would be about 1,500-ft. A profile is shown on Figure XI-17. The total estimated 1980 present worth cost of the new conduit is $2,000,000.

## Non-Structural Alternatives

Non-structural alternatives consist of a dry weather overflow (DWO) abatement program, and the development of a continuous sewerage system maintenance program (i.e., best management practices (BMP's). These items are considered necessary for the proper operation of the system, and to reduce the amount of pollution entering Dorchester Bay from non-point sources. Results of field investigations revealed several maintenance-related problems within the sewerage system, that resulted in dry weather overflows (Chapters IV and V). It is likely that several other problem areas and sources of DWO's may exist within the study area, thus investigatory work as well as continuous maintenance is required to locate and remedy these sources of pollution.

Dry Weather Overflow Abatement Program. As discussed in Chapter V, several occurrences of direct dry weather discharge were discovered in the study area, caused primarily by blockages within regulators. These occurrences resulted in an estimated 0.77 mgd of DWO's discharging to the bay (Table V-5). Additional and undiscovered system operational problems and improper sanitary connections to storm drains may exist, and should be located and corrected. The expected improvement to the water quality of the bay if DWO's are eliminated was discussed in Chapter IX. Table IX-4




NOTE: FOR A PLAN VIEW OF THIS POSSIBLE ROUTE SEE FIG. X-11

FIG. XI-17 PROFILE OF POSSIBLE ROUTE OF PINE NECK CREEK STORM DRAIN RELOCATION

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in that chapter shows that under the existing situation, all beaches within the study area, except for Pleasure Bay, contravene the SB total coliform standards. However, if DWO's were eliminated, all beaches, except for Tenean, would meet the standards. Thus, for a relatively minor cost that is primarily labor intensive, the increase to the long-term water quality of the bay is significant.

The recommended DWO abatement program includes the following:

- Inspection of each manhole and regulator within the study area and the removal of blockages and/or the correction of structural problems.

- Inspection of all storm drains that convey runoff directly to the bay, and the identification and correction of improper, dry weather flow connections - dye testing would be done as required to determine the source of these discharges.

- Recommend areas and/or specific regulators and manholes where continuous maintenance is required for proper system operation.

It is estimated that two-three man crews would be required to perform these work tasks over a 1-year period. The estimated 1980 present worth cost of this program is $100,000, and is included as part of the total recommended plan.

Best Management Practices (BMP's). As discussed in Chapter X, the following best management practices were selected for detailed evaluation:

- regulator maintenance

- catchbasin cleaning and maintenance

- sewer cleaning

- tide gate repair and maintenance

- existing in-system storage

- legislation

Regulator maintenance and tidegate repair and maintenance are considered imperative to insure proper system operation and the prevention of DWO's. System operation problems would be identified in the DWO abatement program, and these items would be inspected and maintained continuously. Tide gate inspections are important, since improper closure of the gates may result in surcharge conditions in upstream sewers, possibly resulting in DWO's. Catch basin cleaning and maintenance would enchance the systems hydraulic operation, and also result in a probable reduction in the amount of nutrients entering the bay, as discussed in Chapter X. Sewer cleaning would also enhance system operation by eliminating

restrictions in system conduits, and increasing the carrying capacity of heavily silted sewers. Sewers having chronic sediment build-up problems (i.e., Dorchester interceptor as discussed in Chapter IV) would be identified and cleaned regularly.

It is estimated that the above system-related BMP's would require one, three-man crew assigned full time to the Dorchester Bay study area. This crew would be fully equipped with maintenance and repair equipment, and would have access to sewer cleaning equipment when required. The total estimated 1980 present worth cost of this program including equipment, labor and sediment disposal is $420,000 for the entire study area.

In-system storage has been evaluated as part of the storage/ containment alternatives discussed earlier in this chapter. Results indicate that the utilization of existing, available in-system storage is cost-effective when compared to off-line facilities for the Dorchester portion of the study area. However, the concept of screening/disinfection has been recommended for the Dorchester area, since it proved to be more cost-effective than in-system or off-line storage.

Legislation is not required to implement the above best management practices, unless responsibility for certain items (i.e., catchbasin cleaning) is transferred from the City of Boston to the MDC. A discussion regarding the need for legislation in order to implement the recomemnded plan is included in Chapter XIII.

## Summary

The results of the detailed evaluation of alternatives enabled the development of a selected program for improving the water quality of Dorchester Bay. This program includes the recommended structural and non-structural alternatives that were determined to be the most cost-effective and environmentally sound solutions to the combined sewer overflow, as well as the dry weather overflow problems. The items included in the program and their associated estimated 1980 present worth costs are summarized in Table XI-2.

TABLE XI-2 SUMMARY OF SELECTED PROGRAM

| | ITEM | ESTIMATED 1980 PRESENT WORTH COST |
|---|---|---|
| 1. | Dry Weather Overflow Abatement Program | $ 100,000 |
| 2. | Hoyt Street Regulator Modifications | 65,000 |
| 3. | Pine Neck Creek Storm Drain Relocation | 2,000,000 |
| 4. | South Boston Recommendations<br>. Storage/containment facility | 17,800,000 |
| 5. | Dorchester-Fox Point Recommendations<br>. Screening/disinfection facility | 3,880,000 |
| 6. | Dorchester-Commercial Point Recommendations<br>. Screening/disinfection facility | 3,080,000 |
| 7. | Best Management Practices | 420,000 |
| | TOTAL | $27,345,000 |

# CHAPTER XII - RECOMMENDED PLAN

## General

Presented in this chapter is a detailed description and preliminary design of the recommended plan. The plan is comprised of the major components discussed in Chapter XI, and miscellaneous recommendations required to improve the operation of the existing combined sewer system. The recommended plan's components are outlined below:

### Structural Recommendations

South Boston Subarea:
- storage/containment facility and consolidation conduit

Dorchester Subarea:
- Hoyt Street regulator modifications
- Commercial Point screening/disinfection facility
- Fox Point screening/disinfection facility
- Relocation of the Pine Neck Creek separate storm drainage system's outlet

### Non-Structural Recommendations

Total Study Area:
- Dry weather overflow abatement program
- Best management practices

### Miscellaneous Recommendations

Total Study Area:
- Cleaning of CSO conduits
- Dredging at CSO outlets
- BOS-090 CSO outlet

For the structural recommendations, design criteria, proposed mode of facility operation, preliminary design drawings, and total estimated costs (i.e., capital construction plus annual operation and maintenance costs) are presented. The non-structural and miscellaneous recommendations are described in detail, and the total estimated costs are presented. Management considerations regarding the residual wastes of the recommended plan, and Dorchester Bay's expected water quality improvements by implementation of the recommended plan are discussed in the final sections of the chapter.

The proposed staging and implementation schedule of the recommended plan, along with the identified legal and institutional issues, are presented in Chapter XIII.

Volume II - Chapter V presents the environmental issues relating to the recommended plan.

## Preliminary Design of Structural Recommendations

### General

The recommended structural facilities are presented in this section by service area (i.e., South Boston and Dorchester). As shown previously on Figure XI-I in Chapter XI, the South Boston area includes the areas tributary to CSO outlets BOS-080 through BOS-087. The Dorchester area includes the areas tributary to BOS-088/089 (Fox Point), BOS-090 (Commercial Point), and the Pine Neck Creek separate storm drain.

For this final phase of the facilities plan, the RUNOFF and TRANSPORT "blocks" of the computer model SWMM, in conjunction with the unified interceptor model, were used to generate data on the hydraulic response of the sewerage system for a selected design storm event. This data was used in sizing structural facilities, and the design storm events used for the analyses were the 3-month, 4-hour, and 1-year, 6-hour synthetic hyetographs, as developed by the Charles River Basin consultant, were used as rainfall input data. The 3-month frequency storm was used in the preliminary design of the South Boston storage/containment facilities, with the control level of the resulting CSO volumes representing 4 overflow events per year (i.e., 1 event per bathing season.) This level of control was determined in Chapter XI to be the most cost-effective for the South Boston portion of the study area. For the Dorchester screening/disinfection facilities, the 1-year frequency storm was selected for hydraulic design. A discussion regarding the selection of this design storm is presented later in this chapter.

A detailed discussion of the model SWMM was presented in Chapter VII, and a description of the unified interceptor model is included in Appendix B. The development of the synthetic design storm events, including the hyetographs, is presented in the Charles River Basin Facilities Plan. Subsequent sections of this chapter include discussions on how these modeling tools were utilized in the preliminary design of structural facilities. A detailed description of each facility including design criteria, site considerations, preliminary drawings, and total estimated costs are also presented.

### South Boston

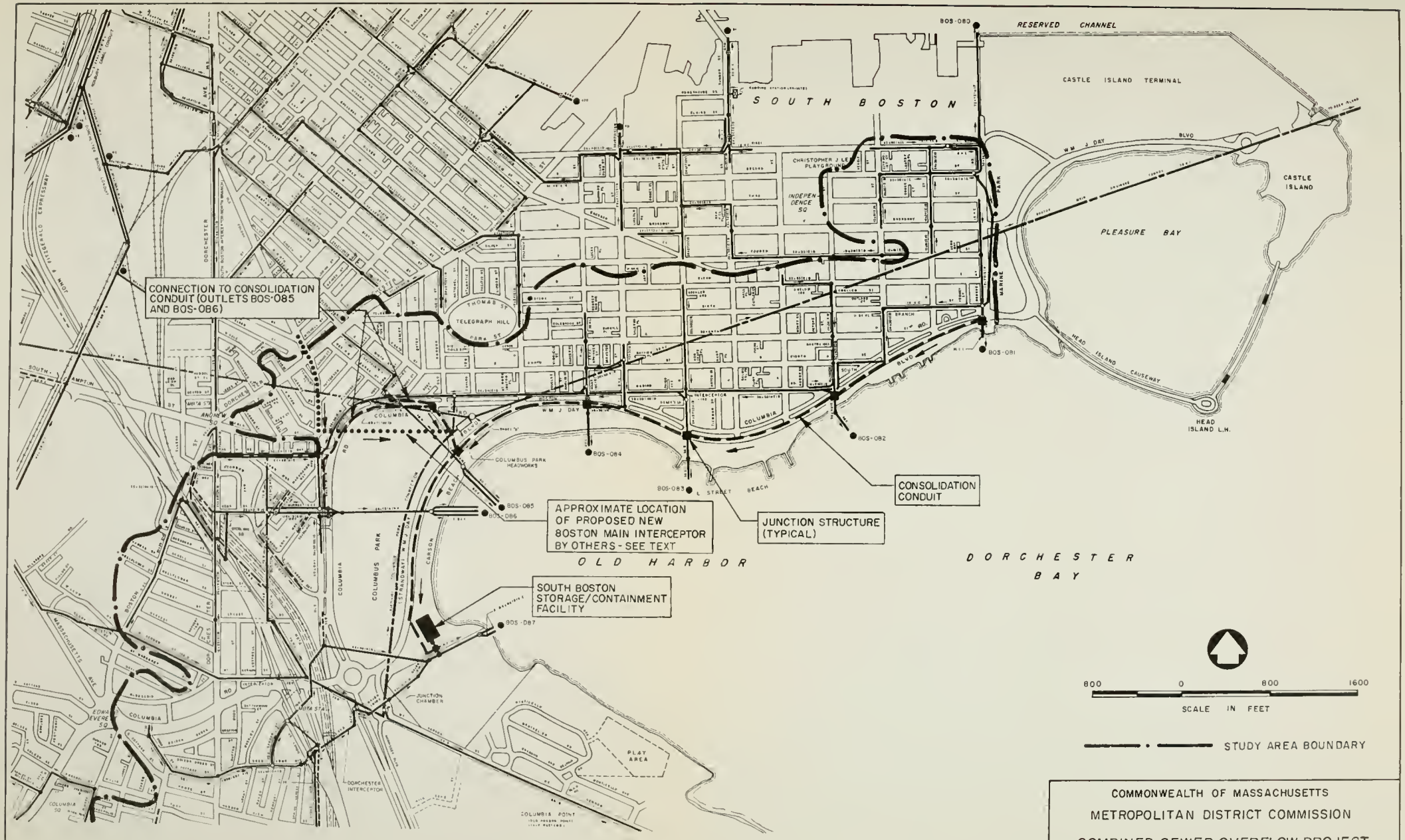
Structural recommendations for the South Boston portion of the study area consist of storage/containment facilities. As shown on Figure XII-1, components of these facilities include a consolidation conduit for interception of CSO's generated from the area tributary to outlets BOS-081 through BOS-087 (543 acres), and an underground storage tank (with associated superstructure) for containment of CSO's during wet weather periods. The stored CSO's would be pumped back to the existing system when capacity becomes available. Including the CSO's generated from the 47 acre area tributary to outlet BOS-080 in the storage/containment facilities is not considered to be warranted. A further discussion of this determination is presented later in this chapter.

The unified interceptor model was used in conjunction with SWMM to generate design storm flows used in sizing the consolidation conduit and storage facility. Two interceptor model networks were developed, representing (1) the existing interceptor system, and (2) the proposed interceptor system which includes replacement of the East Side interceptor (north and south branch) and the Boston main interceptor. A complete discussion of the proposed new interceptor system is included in the Inner Harbor Area Facilities Plan. A brief discussion of those components of the new system that impact on the hydraulics and flow patterns of the South Boston portion of the Dorchester Bay study area is included in the subsequent section. The use of the interceptor model, including how the proposed interceptor improvements were simulated, is also discussed.

Use of the Unified Interceptor Model. The unified interceptor model was used to generate boundary condition hydrographs for each design storm event at the three locations where interceptors from adjacent study areas enter the Dorchester Bay study area. These locations are designated as A, B and C on Figure 2 of Appendix B, and they represent where the Dorchester, Boston Main, and South Boston (north branch) interceptors, respectively, enter the Dorchester Bay study area under existing conditions. The second interceptor model network represents the proposed future interceptor system tributary to the Deer Island treatment plant. These interceptor system improvements are under the jurisdiction of the Boston Water and Sewer Commission, and portions are currently in the final design phase.

As shown on Figure XII-1 and schematically on Figure 2A of Appendix B, the proposed new Boston Main interceptor (BMI) would intercept both the north and south branches of the South Boston interceptor (SBI) prior to entering the Columbus Park headworks. Accordingly, the portions of the north and south branches of the SBI located downstream of the connections to the new BMI would accept only local dry weather flows. The existing BMI would be abandoned, except for the section downstream of the SBI connection. This portion of the BMI would primarily serve to convey flows from the SBI (local flows only as discussed above) and from the Dorchester interceptor, to the Columbus Park connection for transport to the Columbus Park headworks.

As presently conceived, hydraulic relief of the new interceptor system would be provided by a dynamically operated regulator located near the upstream end of the new BMI. This would serve to direct excess wet weather flow from the proposed new East Side interceptor (north branch) and the Dorchester Brook Sewer to the proposed Fort Point Channel CSO treatment facility which is included in the Inner Harbor Area's recommended plan. Currently, the City of Boston's Calf Pasture pumping station is operated during certain storm events to relieve the Columbus Park headworks of excess wet weather flow. Under the system's proposed improvement program, which in addition to the interceptor replacements also includes a program by the MDC to restore pumping capacity to the original design conditions at the Deer Island pumping station (Appendix B), the Calf Pasture pumping station and the Moon Island retention facilities would eventually be phased out of service.

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RECOMMENDED STRUCTURAL FACILITIES
FOR SOUTH BOSTON
FIG. XII - 1

The diversion of excess wet weather flow to the Inner Harbor area's proposed Fort Point Channel facility was simulted when modeling future conditions. The unified interceptor model simulated a regulator device which would divert wet weather flow from the north branch of the East Side interceptor in order to maintain a total flow of 182 mgd or less at the Columbus Park headworks. This flow represents the hydraulic design capacity of the headworks with one of the four channels out of service. The 182 mgd total includes a temporary peak capacity allocation of 70 mgd for the Dorchester Bay and Neponset River study areas. The regulator device will be an important hydraulic control in the future interceptor system. The design of this structure is described in the Inner Harbor area facilities plan.

The boundary conditions estimated by the unified interceptor model in the form of stage and discharge hydrographs, were used as input to the South Boston SWMM model. The SWMM model was then used to generate CSO flow rates and volumes for the design storm events. This information was used in the preliminary design of the consolidation conduit and storage/containment facility.

Hydraulic Design Criteria. As discussed in Chapter XI, during the detailed evaluation phase, the long-term statistical model SEMSTORM was employed to estimate consolidation conduit flow rates and required off-line storage volumes. Based on a South Boston sewerage system conveyance capacity that was estimated by area-weighting the average of the historical maximum daily Columbus Park headworks flow rates, and an estimated available in-system storage volume, the required off-line storage volume for the 4-overflow days-per-year control level was estimated to be about 8.8 mil. gal. (Alternative SB-B in Chapter XI). During the preliminary design phase, the model SWMM was used, in conjunction with the unified interceptor model, to generate CSO flow rates and volumes for a single, design storm event.

A schematic representation of the consolidation conduit and storage facility was added to the South Boston SWMM network, and the model was run for the 3-month, 4-hour design storm. Input hydrographs from the Inner Harbor study area and from the Dorchester portion of the Dorchester Bay study area were obtained by running the "future condition" unified interceptor model for the design storm. Results of this initial SWMM run indicated that the required off-line storage volume is in the order of 1.5 mil. gal., which is considerably less than 8.8 mil. gal. estimated during the detailed evaluation phase using SEMSTORM. Examination of SWMM output data revealed that the system's conveyance capacity is significantly affected by the boundary conditions from the interceptor model. The SWMM model, unlike SEMSTORM, can dynamically simulate system hydraulics during a storm event. SWMM demonstrated that a much greater conveyance capacity is available than the rate previously estimated and used for SEMSTORM. Other factors that contribute to the lower, required off-line storage volume are as follows:

- The boundary conditions from the "future condition" unified interceptor model were based on full use of the maximum hydraulic capacity of the Columbus Park headworks (180 mgd) which assumes restoration of original pumping capacity at Deer Island.

- o Since hydraulic relief of the proposed new interceptor system is located upstream of the South Boston area (in lieu of within the South Boston area as is the existing Calf Pasture pumping station) the resulting input hydrographs at the boundaries contribute less flow to the South Boston system and have smaller, flatter peaks than the hydrographs from the existing condition runs. Thus, more treatment capacity at the headworks is available for the South Boston area under the future condition.

- o Since the new Boston Main interceptor (BMI) will intercept up to the peak dry weather flows from both the north and south branches of the South Boston interceptor (SBI) and provide conveyance directly to the Columbus Park headworks, additional in-system storage becomes available in the portion of the existing interceptor system located downstream of these connections.

In order to determine the sensitivity of the South Boston sewerage system to the new BMI flows, the 3-month 4-hour design storm input hydrograph from the unified interceptor model representing flows from the new BMI was replaced with an alternate hydrograph representing a "worst-case" condition. This hydrograph had approximately the same shape and volume of flow as the design storm hydrograph, however, the time of peak flow corresponds to the peak flow in the Columbus Park connection, which consists primarily of the flow from the Dorchester interceptor (under the proposed new interceptor system as discussed earlier in this chapter). When using the design storm input hydrographs, the peak occurred approximately 30-minutes apart. Utilization of this alternate hydrograph resulted in a greater backwater condition in the south branch of the SBI, increasing the overflow volume and, accordingly, the required off-line storage volume, to about 2.0 mil. gal.

Several additional runs were made representing various combinations of sewerage system configurations and mode of operation under varying storm conditions in order to assess their impacts on the required off-line storage volumes. Evaluated were the following:

- o Start pumpback from the storage tank to the system at the end of the storm event, instead of when headwork's capacity becomes available during the storm event,

- o Eliminate the regulator located on the south branch of the SBI which provides hydraulic relief by conveying excess flow directly to the Columbus Park connection, just upstream of the headworks;

- o Provide for sediment deposition in the trunk section of the SBI and the Columbus Park connection (no deposition was observed in the south branch of the SBI during field investigations, therefore, this conduit was assumed to be"clean");

- o Simulated back-to-back storms, by running SWMM with the 3-month, 4-hour synthetic design storm hyetograph followed by the one-month, 4-hour synthetic storm hyetograph, and assuming pumpback starting when headworks capacity becomes available;

- o Existing system conditions (i.e., assuming no new interceptor improvements ) under the above four cases (i.e., four separate runs).

The resulting required off-line storage volumes for the above runs ranged between 1.5 and 2.0 mil. gal. Therefore, it was concluded that the various storm and system conditions simulated, did not significantly affect CSO volumes. Since overflows occur at the regulators within tne combined sewer system, interceptor flow conditions would affect the volume of CSO's only during extreme interceptor surcharge conditions when the hydraulic grade line elevation restricts flow from the regular connections. This occurred when the alternate "worst-case" BMI input hydrograph was utilized, and the required CSO storage volume was increased from 1.5 to 2.0 mil. gal. The increase was minimal because the surcharge condition in the SBI occurred only during peak flow conditions. Based on the above, modeling, it was concluded that an off-line storage volume of about 2.0 mil. gal. would be required. However, considering the inherent inaccuracies associated with the modeling tools, a 20 percent contingency factor of 0.4 mil. gal. was added to this volume, resulting in a total estimated offline storage volume of about 2.4 mil. gal.

As discussed earlier in this chapter, the unified interceptor model network employs several assumptions, primarily relating to future system operation. These include:

- o The availability, when required, of the maximum hydraulic capacity of the Columbus Park headworks, which is based on full restoration of Deer Island's original pumping capacity;

- o Interceptor system relief would occur at a regulator located at the upstream end of the new BMI;

- o The availability of the maximum conveyance capacity of existing and proposed conduits, which is based on "textbook" Manning "n" values and no sediment deposition in the conduits;

- o All regulators, tide gates and other sewerage system appurtenances are functioning properly, thus allowing full use of the system's maximum flow conveyance ability.

To allow for a potential reduction in the future system's conveyance capacity, and for any deviations to the currently proposed operating policy of the new interceptor system, primarily regarding hydraulic relief, it was considered appropriate to increase the required amount off-line storage by a "factor-of-safety". For the purpose of determining the magnitude of this increase, hand computations were performed to estimate the volume of CSO generated by the 3-month, 4-hour synthetic design storm. Calculated in-system storage volumes and the South Boston sewerage system's average conveyance capacity from SWMM were utilized in this analysis. Since dynamic

system conditions and flow routing are not taken into account with hand calculations, this method resulted in a conservative required storage volume estimate of about 3.5 mil. gal. Considering that the computer model analysis resulted in a volume of about 2.4 mil. gal., a final required off-line storage volume of 3.0 mil. gal. was adopted for this facilities plan. This volume should have a sufficient "factor of safety" to allow for inherent modeling inaccuracies, improper system operation, and/or significant deviations from the assumptions incorporated into the modeling tools.

Design flowrates for the proposed consolidation conduit were determined by a similar method. For modeling purposes, an "oversized" conduit was incorporated into the SWMM network in order to assure adequate conveyance capacity to the storage tank. The peak rate of flow in each reach of conduit (i.e., each section between CSO outlet interception points), which represents the design flowrate, was estimated by running the model for the 3-month, 4-hour design storm. An independent analysis using the rainfall/runoff Rational formula (Q=CiA) was also performed to estimate peak design flowrates for each reach. Results of both methods were then compared. The latter analysis resulted in greater and more conservative flowrates, in the order to 10 to 20 percent, primarily because this method did not take into account the varying flow rates and the routing of the flows through the conduits during the storm events. The higher peak rates were adopted for preliminary design in order to acount for modeling inaccuracies, improper system operation and/or significant deviation to any of the assumptions incorporated into the model. Referring to the consolidation conduits route shown on Figure XII-1, these preliminary design flowrates are summarized as follows:

| Consolidation Conduit Reach | Tributary Area, acres | Preliminary Design Flowrate, mgd |
|---|---|---|
| BOS-081 to BOS-082 | 27 | 12.4 |
| BOS-082 to BOS-083 | 99 | 30.7 |
| BOS-083 to BOS-084 | 128 | 34.7 |
| BOS-084 to BOS-085/086* | 171 | 41.7 |
| BOS-085/086 to storage tank | 353 | 86.8 |
| BOS-087 to storage tank | 190 | 62.4 |

CSO flow rates less than or equal to the above rates would enter the proposed consolidation conduit at the junction structures, which would also serve to provide hydraulic relief for flows that exceed these rates. A description of these structures, including preliminary drawings, is presented later in this chapter.

*CSO's from the areas tributary to these outlets would enter the consolidation conduit at a single location as discussed in detail later in this chapter.

Siting. From an engineering standpoint, the preferred site for the storage/containment facility is located in the northeast portion of Columbus Park, as shown on Figure XI-2 in Chapter XI. Because of its centralized location and proximity to the Columbus Park headworks, this site proved to be the most cost effective as well as the most feasible when considering operation and maintenance requirements. During the preliminary design phase, discussions were held with the City of Boston Parks and Recreation Department, who is the owner of the park. The department's approval was requested, and comments and suggestions, primarily relating to possible multi-use of the required superstructure, were solicited. Department officials were strongly opposed to the construction of the storage/containment facility within the park, and it was suggested that alternate sites be evaluated. Their concerns were based on the expected temporary disruption of this intensely used area of the park during construction, and the permanent loss of parkland resulting from the superstructure. The department felt that since recreational facilities are limited in this area of the City, and because Columbus Park provides the only major recreational facilities for residents of South Boston and portions of Dorchester, any planned disruption would definitely result in public and community opposition. This oppostion was evident from the South Boston Resident's Group, Inc., who expressed their displeasure with the park site for reasons similar to those of the Parks and Recreation Department, and suggested an alternative site be evaluated further south, near Columbia Point. Accordingly, in order to respond to the concerns of the area's residents as well as the City's Parks and Recreation Department, the following three alternative sites were investigated:

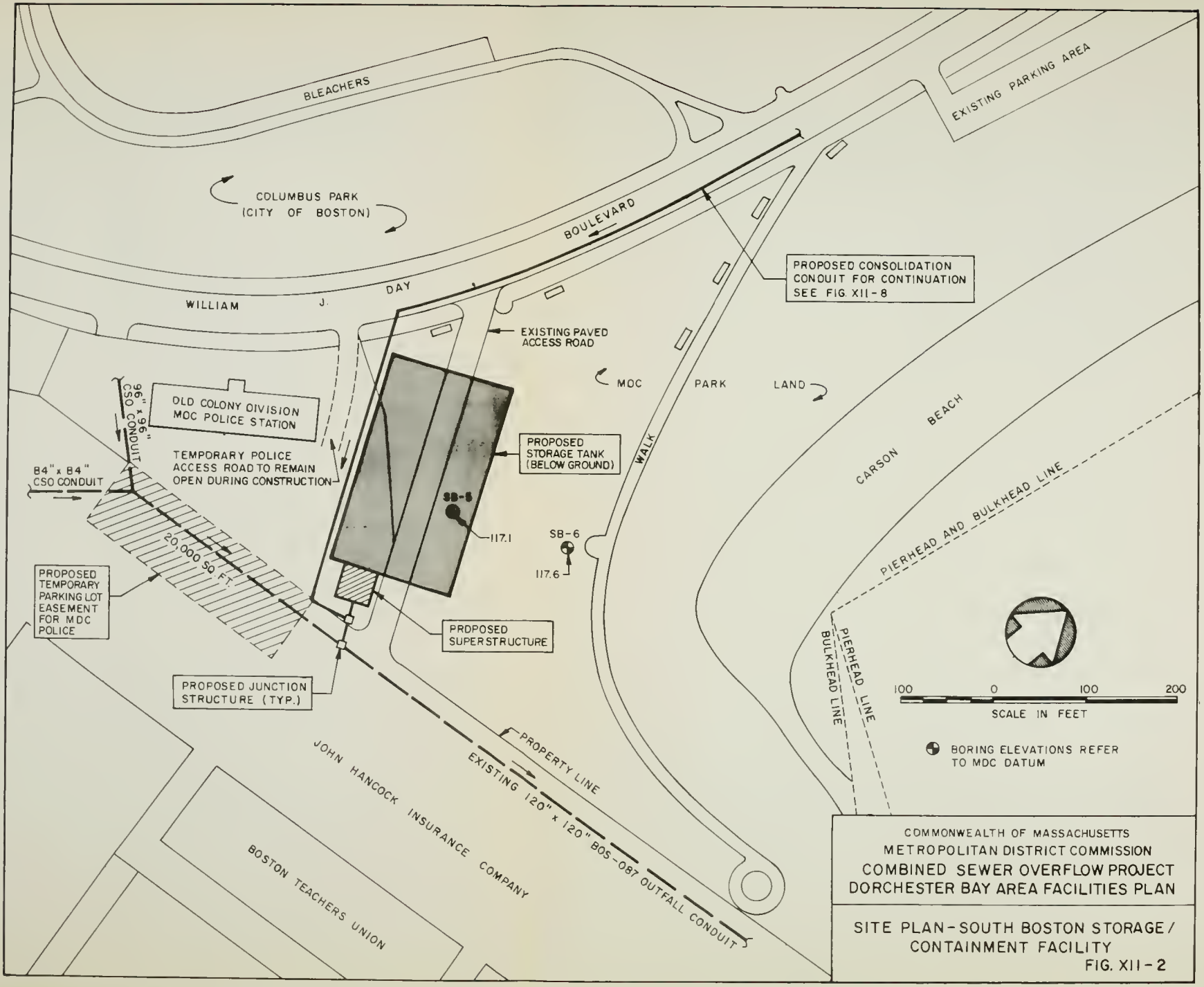
- o Columbus Park headworks
- o Boston Teachers Union parking lot
- o East of the MDC Police Station

The Columbus Park headworks site is located adjacent to the headworks on the triangular-shaped MDC-owned parcel bounded by Columbia Road on the north, Day Boulevard on the south and east and the street connecting these roadways on the west (Figure XII-1). The maximum surface area available for construction of the proposed storage tank at this site is about 20,000 sq. ft. located at the southern tip of this parcel, which includes a portion of the roadway connecting Day Boulevard to Columbia Road. This would require a tank with an effective depth of about 20 ft in order to provide the required 3.0 mil. gal. of storage. The total nominal depth of the tank would be about 26 ft. to provide space for access and for equipment removal mechanisms. Since the invert of the consoliation conduit at this location would be about 23-ft. below ground elevation, the bottom of the tank would be about 41-ft. below grade, requiring an excavation of about 45-ft. depth. In order to excavate to this depth, a minimum work area of about 40-ft., and a preferred area of 80-ft, around the periphery of the tank would be required. The work area would not be available at this site. Also, temporary use of a portion of Columbus Park would most likely be required for the storage of equipment and excavated material. Such uses of the parkland would not be desirable for the reasons mentioned earlier. Accordingly, this site was eliminated from future consideration.

The Boston Teachers Union parking lot site is located north of the existing Boston Teachers Union building and south of the MDC-owned parkland ("Mothers Rest") which is located adjacent to Carson Beach. The Teachers Union currently leases this property (with an option to buy) from the John Hancock Insurance Company and utilizes it as a parking area (Figure XII-2). The primary reason for pursuing this site was to avoid any short or long-term disruption to MDC parklands resulting from construction of the facility, and the permanent location of the required superstructure. However, officials from the Boston Teachers Union expressed concern regarding the likely major disruption to a significant portion of their parking area during construction and the permanent loss of parking area due to the superstructure. An additional concern was the possible inconvenience to the patrons of the Teachers Union function rooms during construction in regards to the remote location of temporary parking and the displeasing aesthetic of the area during construction. Accordingly, the Teachers Union officials stated they would strongly oppose the construction of the storage/containment facility in their parking lot.

The possibility of locating the facilities just north of the parking lot and in the MDC-owned parkland and/or property owned by the MDC Police (Old Colony Division) was investigated. Several discussions were held with officials from the MDC Parks and Recreation Division, and from the MDC Police, in order to select the location of the superstructure and tank configuration that is acceptable by both concerns. The mutually agreed upon site is shown on Figure XII-2. As indicated, the superstructure, which consists of a 40-ft x 35-ft x 13-ft high building, would be located on the southeast corner of the MDC Police parcel with the below ground storage tank (230-ft long by 130-ft wide) located beneath portions of the MDC Police parcel, the access road, and the MDC parkland. This location of the superstructure was preferred by the MDC Parks and Recreation Division over a site located at the opposite end of the tank off of Day Boulevard since it would result in minimal adverse visual and aesthetic impacts to the parkland. A portion of the southeast section of the MDC police parcel is presently used for storage of MDC maintenance vehicles. Sufficient area, however, is available for the proposed superstructure as well as for vehicular storage, thus no significant disruption to the present use of this area would result.

The storage tank dimensions and configuration were based on maximum use of the existing access road, which would be temporarily blocked during construction, and maximum utilization of the MDC Police Station parking area to the extent that would not significantly disrupt normal activities. As shown on Figure XII-2, a temporary parking area would be provided for use by the MDC police during the construction period. A temporary access road would be maintained during construction on the northeast side of the police station for vehicular entrance/exit to the parking area. Utilization and, accordingly, temporary disruption of a portion of the MDC parkland would be minimized under this facility arrangement. This area of the park is presently used only for passive recreational activities. During and after most large rainfall events significant ponding of water currently occurs in this area due to a ground depression. After construction, this portion of the park would be regarded to eliminate this ponding problem. It is expected that approximately one acre of the parkland would




COMMONWEALTH OF MASSACHUSETTS
METROPOLITAN DISTRICT COMMISSION
COMBINED SEWER OVERFLOW PROJECT
DORCHESTER BAY AREA FACILITIES PLAN

SITE PLAN - SOUTH BOSTON STORAGE/ CONTAINMENT FACILITY
FIG. XII-2

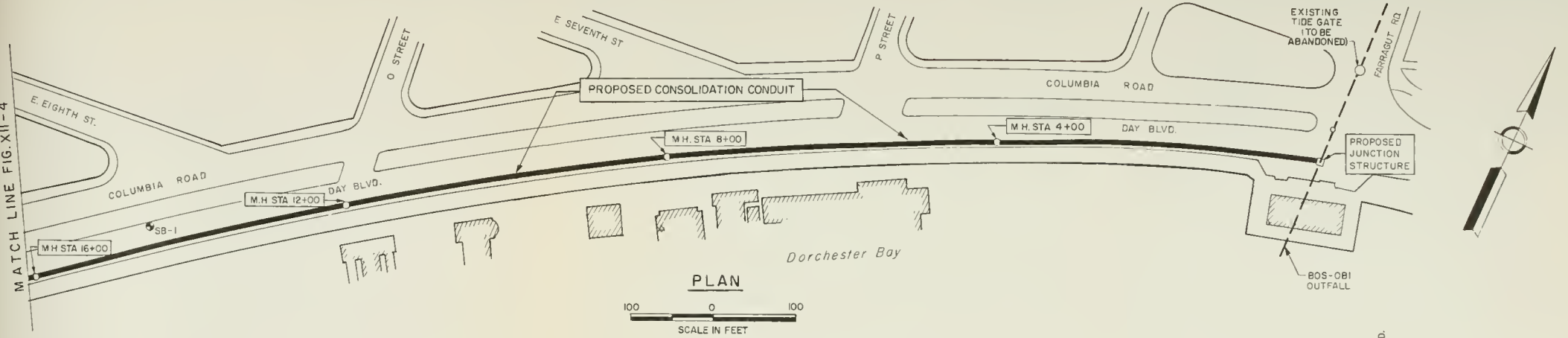
be temporarily utilized during the estimated two year construction period, however, no significant disruption to normal park activities would occur. The existing access road would also be restored after construction, and it would provide access to the superstructure for maintenance purposes and the removal of screenings, as well as serving as access to the Boston Teachers Union parking lot. The MDC Police Station parking lot and other disrupted areas of their parcel would, at a minimum, be restored to existing conditions.

Consolidation Conduit. The proposed consolidation conduit would serve to convey CSO's from outfalls BOS-081 through BOS-087 to the proposed storage/containment facility. The conduit and junction structures were preliminary designed to intercept and convey CSO's up to the estimated peak rate of flow from the 3-month, 4-hour design storm. These design flows were presented for each reach of conduit previously in this chapter. The locations of the proposed consolidation conduit and the junction structures are shown on Figure XII-1.

General Description and Design Criteria. Figures XII-3 through XII-10 show the plan and profile of the proposed consolidation conduit from Station 0+0 at Farragut Road (BOS-081) to the proposed storage/containment facility near the MDC Police Station (Station 90 + 40). Figures XII-11 through XII-16 show details of the junction structures and existing regulator structure modifications required to divert flow into the proposed facilities. Table XII-1 summarizes the design criteria for each reach of conduit.

As shown on Figures XII-3 to XII-10, almost the entire length (about 8,600 ft) of the proposed conduit would be located beneath Day Boulevard with the last 400-ft located adjacent to the proposed storage facility in the MDC Police Station parcel (Figure XII-2). Based on the results of a field topographic survey of the entire length of Day Boulevard which located all major utilities including existing sewers and regulators, the conduit route shown was selected from the standpoint of tieing into the existing sewerage system downstream of the existing regulators. Traffic disruption could be minimized by constructing the conduit in sections and completing each section including backfill and street restoration prior to constructing the next section. Since steel sheet piling would be driven on both sides of the trench (construction techniques are discussed in detail later in this chapter), the width of the excavation and the extent of roadway disruption would be minimized. Disruption to normal beach and yacht club activities would also be minimized since the sections of the conduit located adjacent to these areas would be constructed during off season months. The total length of the consolidation conduit is 9,040-ft., and the size ranges from 36-in. (BOS-081 to BOS-082 reach) to 78-in. diameter (final reach).

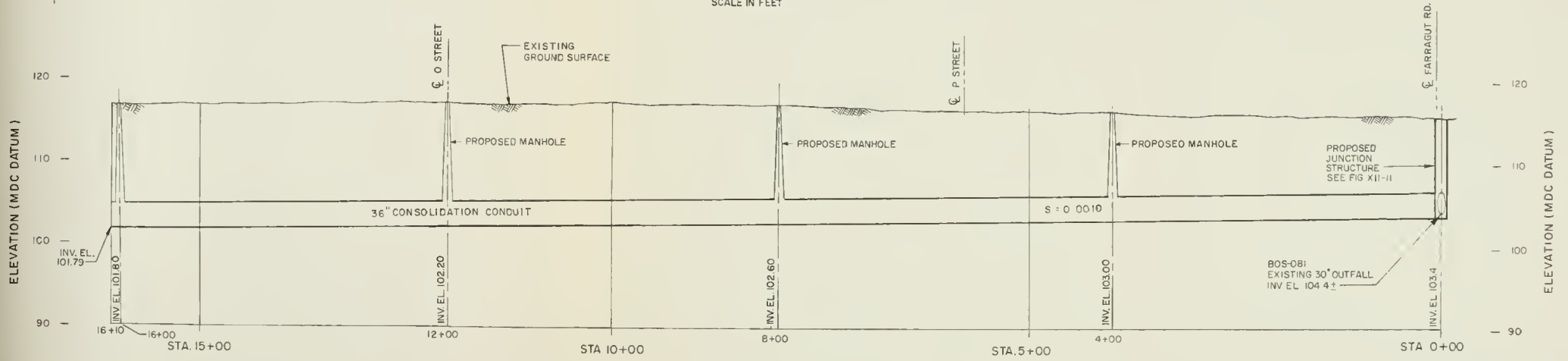
As shown on Figures XII-9 and XII-10, the consolidation conduits' connection with outfalls BOS-085 and BOS-086 extends from the existing regulators on the north and west sides, respectively, of Columbus Park. Constructing a junction structure where each CSO outfall conduit intersects the consolidation conduit in Day Boulevard, as proposed for all other CSO outfalls, was not possible for outfalls BOS-085 and BOS-086 because of tidal influence. Since there are no tide gates on these




PLAN

100 0 100

SCALE IN FEET




PROFILE

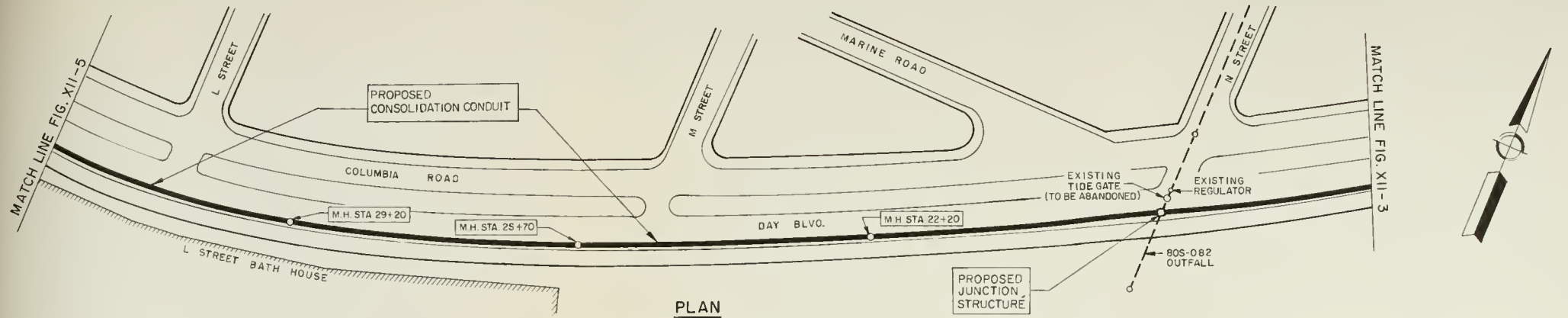
HORIZONTAL SCALE IN FEET 100 0 100

VERTICAL SCALE IN FEET 10 0 10

NOTE · EXISTING UTILITIES NOT SHOWN

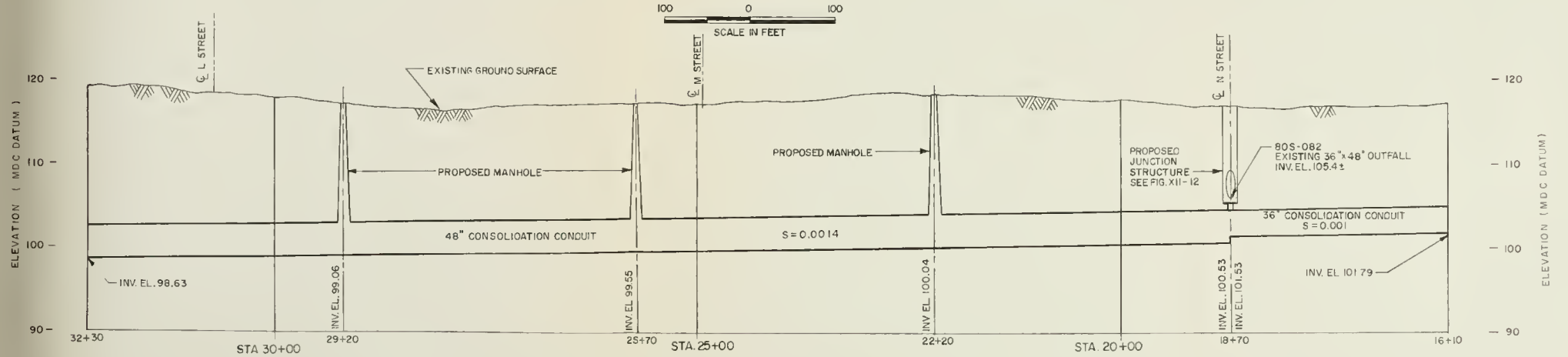
COMMONWEALTH OF MASSACHUSETTS
METROPOLITAN DISTRICT COMMISSION
COMBINED SEWER OVERFLOW PROJECT
DORCHESTER BAY AREA FACILITIES PLAN

SOUTH BOSTON CONSOLIDATION CONDUIT
STA. 0+00 TO STA. 16+10
FIG. XII-3




## PLAN

100 0 100
SCALE IN FEET




NOTE · EXISTING UTILITIES NOT SHOWN

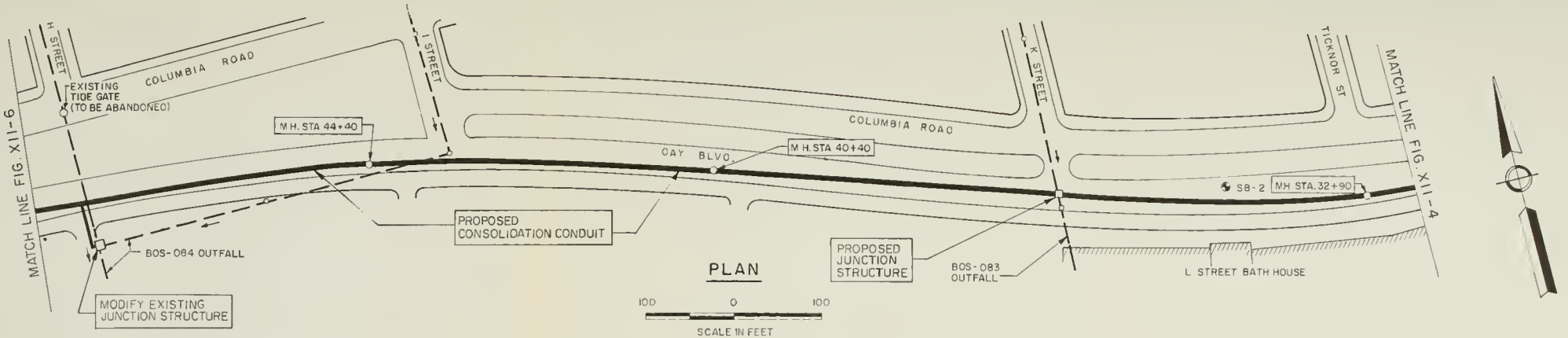
## PROFILE

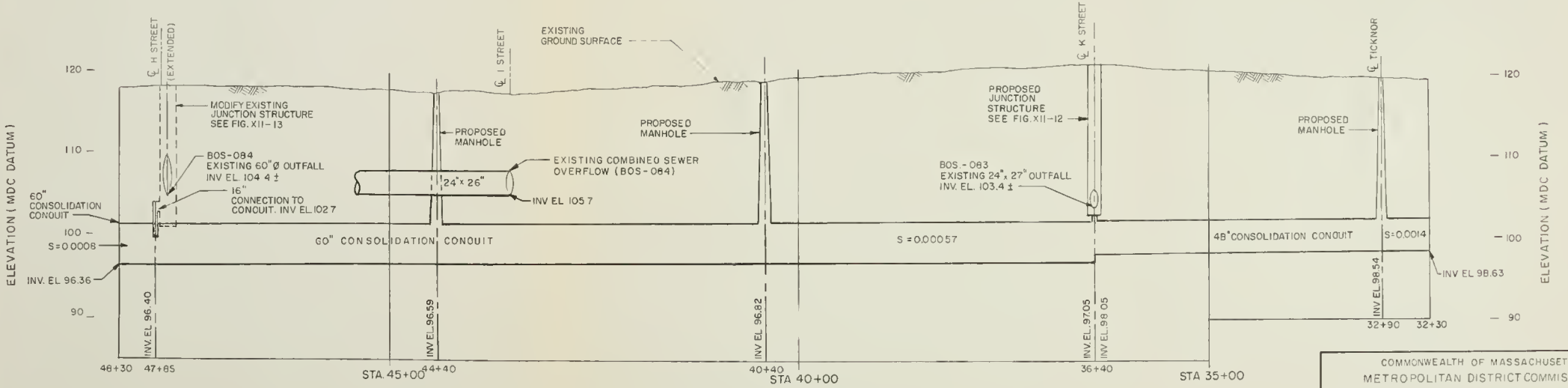
HORIZONTAL SCALE IN FEET 100 0 100

VERTICAL SCALE IN FEET 10 0 10

COMMONWEALTH OF MASSACHUSETTS
METROPOLITAN DISTRICT COMMISSION
COMBINED SEWER OVERFLOW PROJECT
DORCHESTER BAY AREA FACILITIES PLAN

SOUTH BOSTON CONSOLIDATION CONDUIT
STA. 16+10 TO STA. 32+30
FIG. XII-4

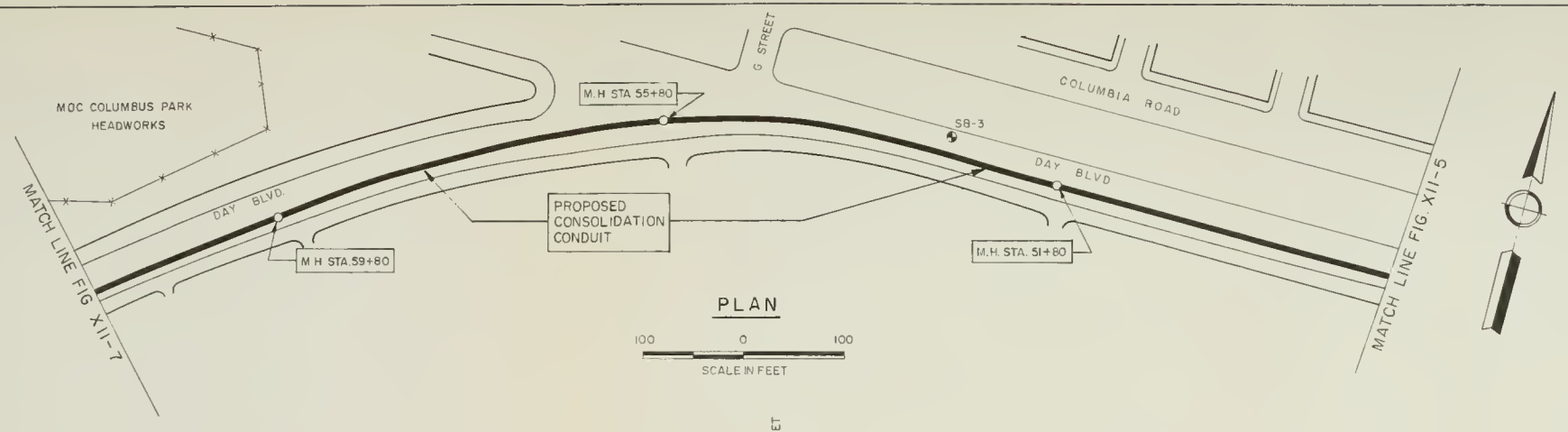



NOTE EXISTING UTILITIES NOT SHOWN

COMMONWEALTH OF MASSACHUSETTS
METROPOLITAN DISTRICT COMMISSION
COMBINED SEWER OVERFLOW PROJECT
DORCHESTER BAY AREA FACILITIES PLAN

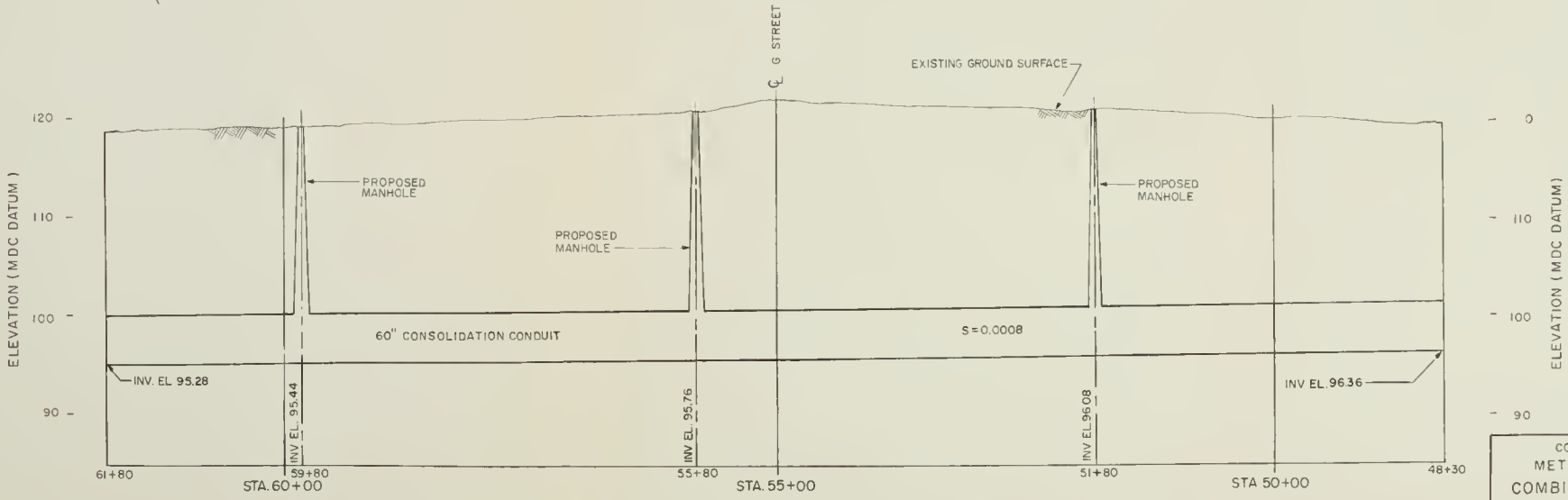
SOUTH BOSTON CONSOLIDATION CONDUIT
STA. 32+30 TO STA. 48+30
FIG. XII-5




PLAN

100 0 100

SCALE IN FEET




PROFILE

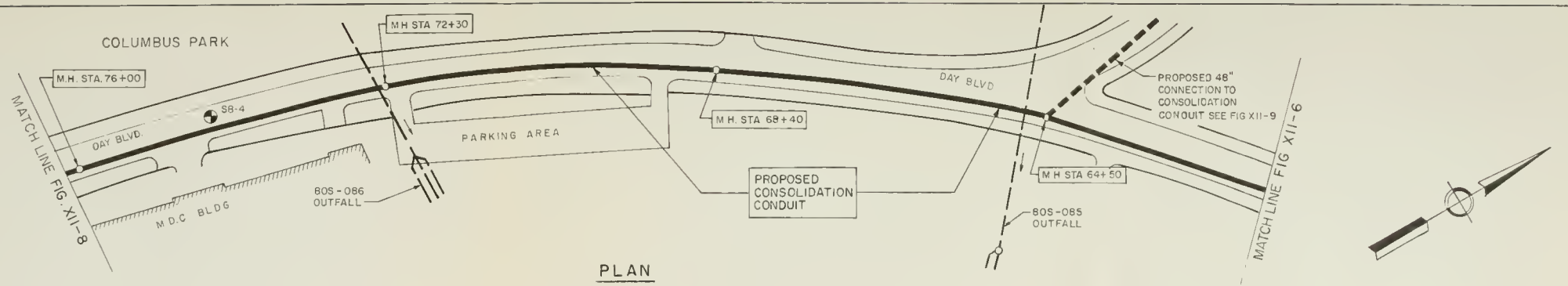
NOTE EXISTING UTILITIES NOT SHOWN

HORIZONTAL SCALE IN FEET 100 0 100

VERTICAL SCALE IN FEET 10 0 10

COMMONWEALTH OF MASSACHUSETTS
METROPOLITAN DISTRICT COMMISSION
COMBINED SEWER OVERFLOW PROJECT
DORCHESTER BAY AREA FACILITIES PLAN

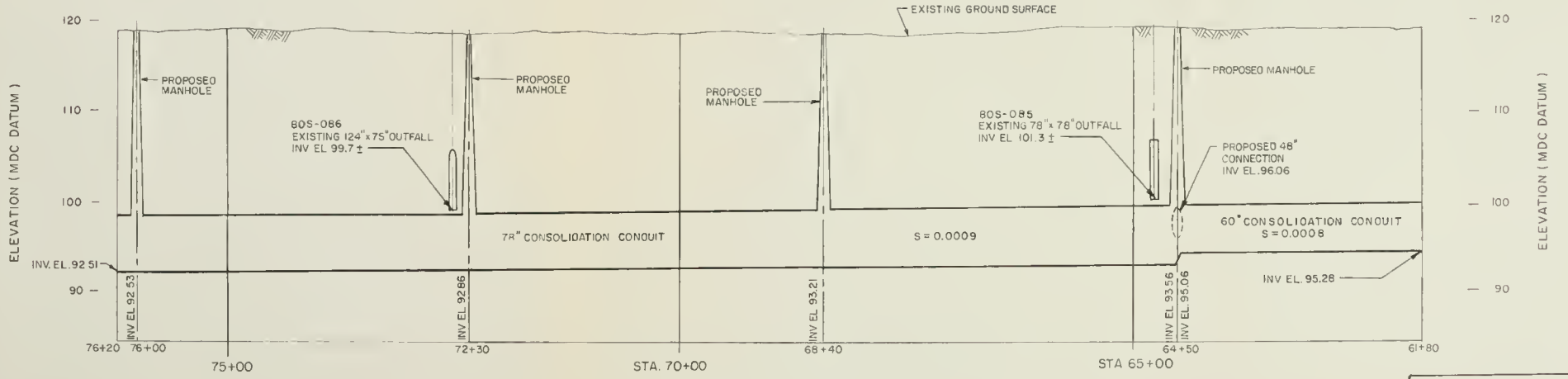
SOUTH BOSTON CONSOLIDATION CONDUIT
STA.48+30 TO STA.61 +80
FIG. XII-6




PLAN

100 0 100

SCALE IN FEET




PROFILE

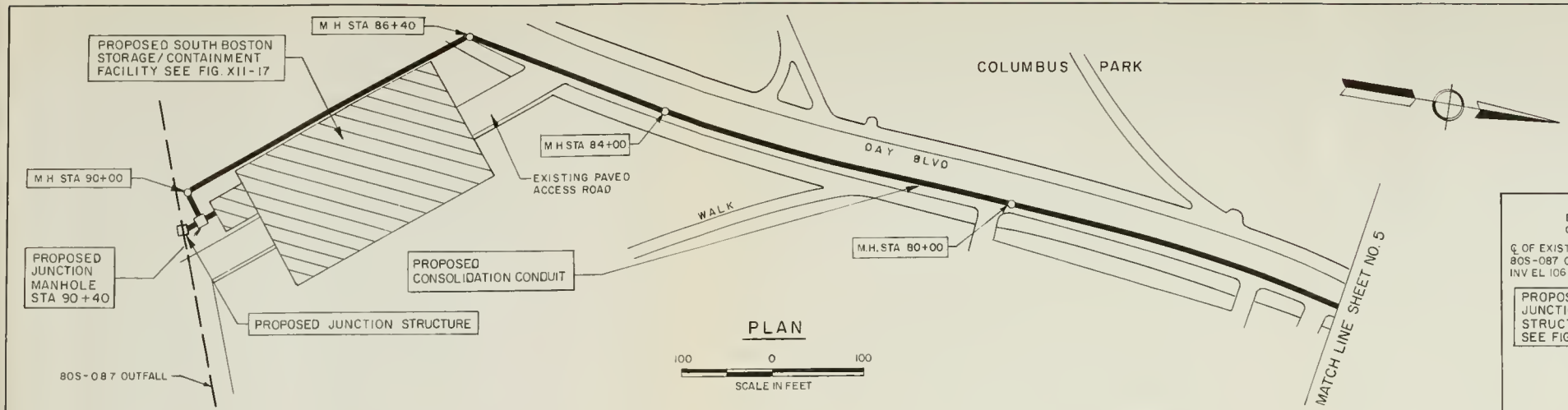
HORIZONTAL SCALE IN FEET 100 0 100

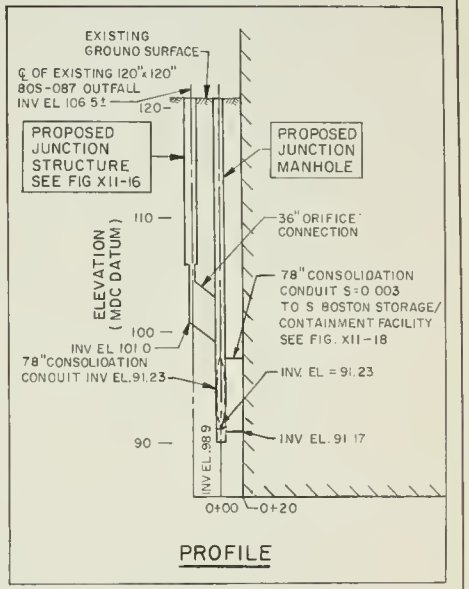
VERTICAL SCALE IN FEET 10 0 10

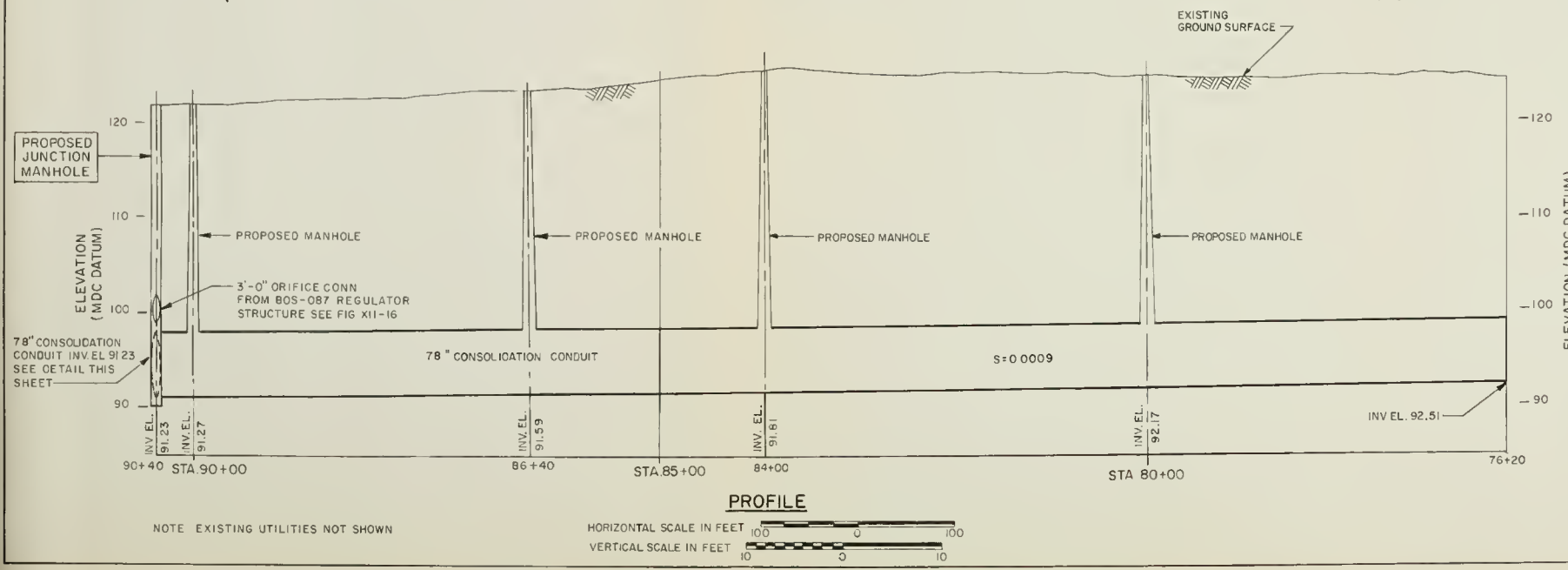
NOTE: EXISTING UTILITIES NOT SHOWN

COMMONWEALTH OF MASSACHUSETTS
METROPOLITAN DISTRICT COMMISSION
COMBINED SEWER OVERFLOW PROJECT
DORCHESTER BAY AREA FACILITIES PLAN

SOUTH BOSTON CONSOLIDATION CONDUIT
STA.61+80 TO STA.76+20
FIG. XII – 7

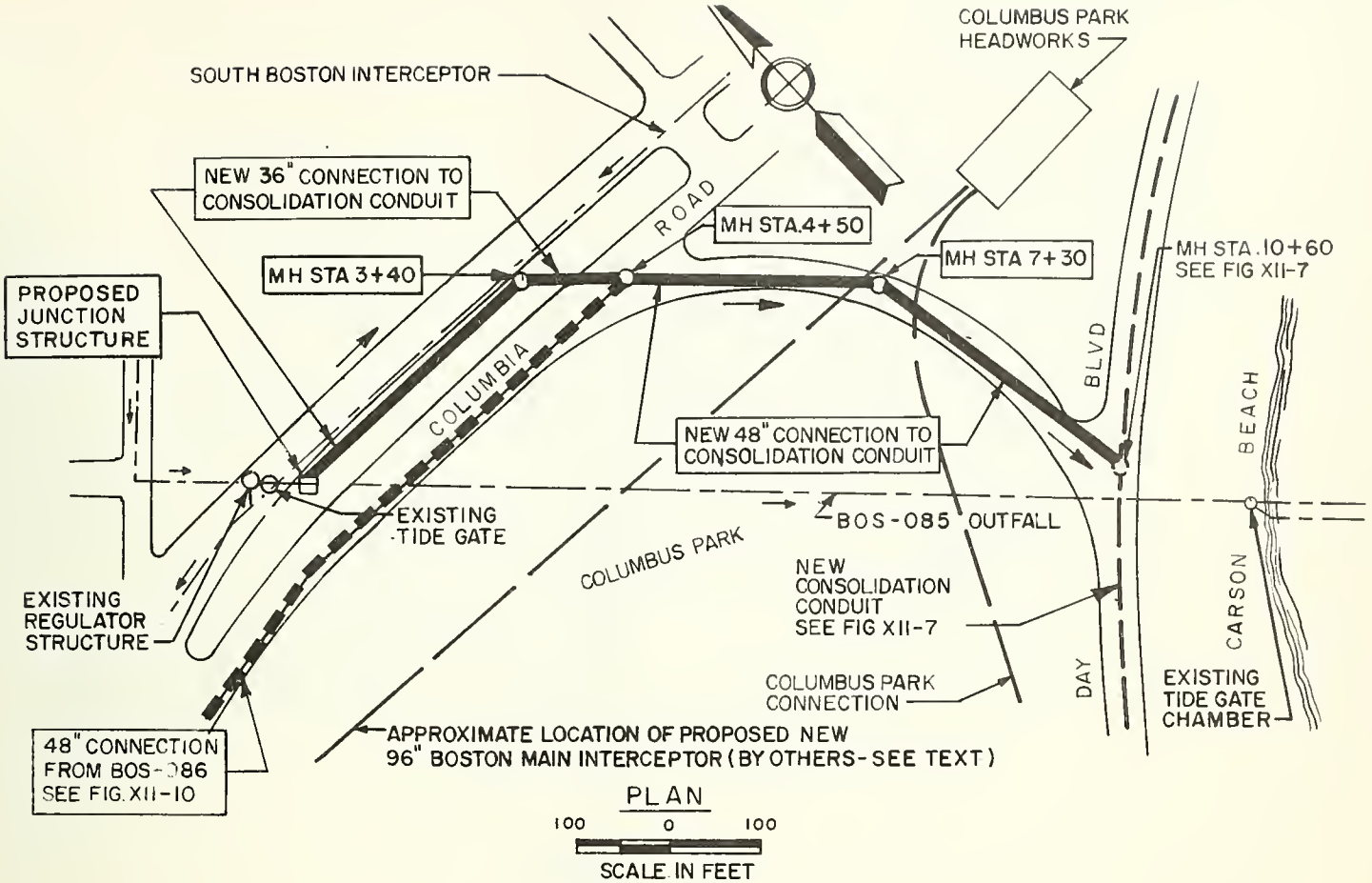


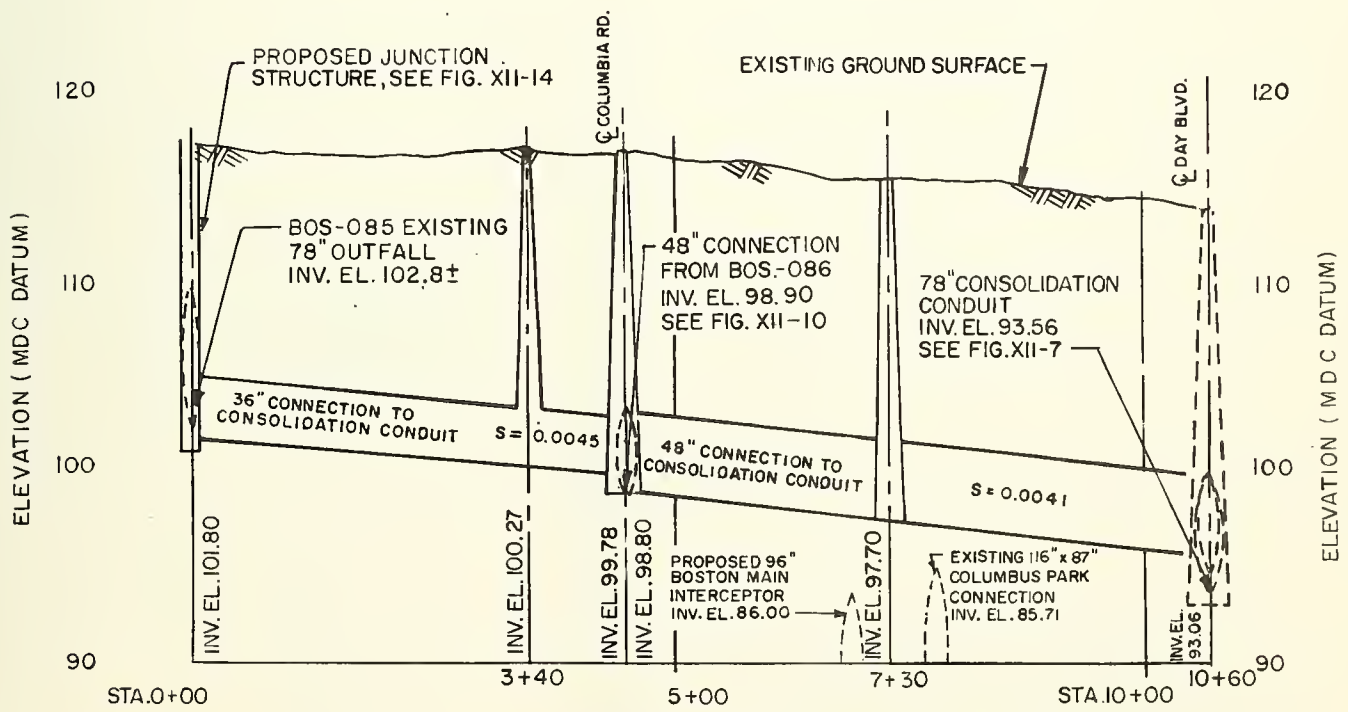




COMMONWEALTH OF MASSACHUSETTS
METROPOLITAN DISTRICT COMMISSION
COMBINED SEWER OVERFLOW PROJECT
DORCHESTER BAY AREA FACILITIES PLAN

SOUTH BOSTON CONSOLIDATION CONDUIT
STA. 76+20 TO STA. 90+40
FIG. XII-8

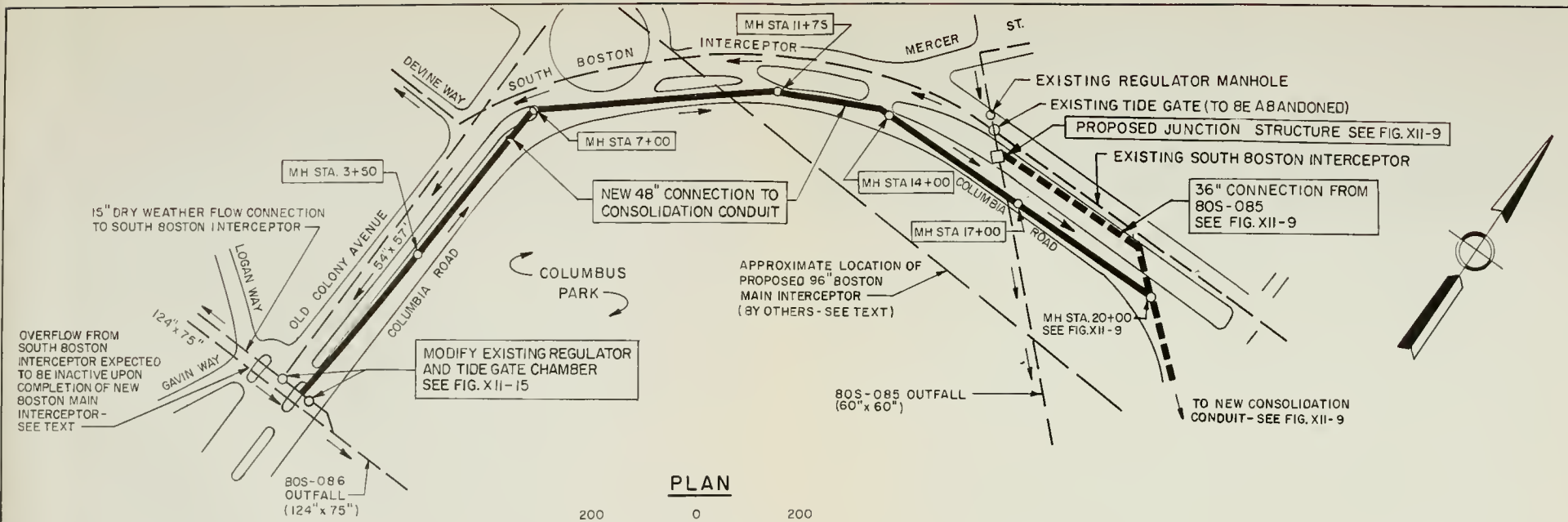



NOTE: EXISTING UTILITIES NOT SHOWN

COMMONWEALTH OF MASSACHUSETTS
METROPOLITAN DISTRICT COMMISSION
COMBINED SEWER OVERFLOW PROJECT
DORCHESTER BAY AREA FACILITIES PLAN

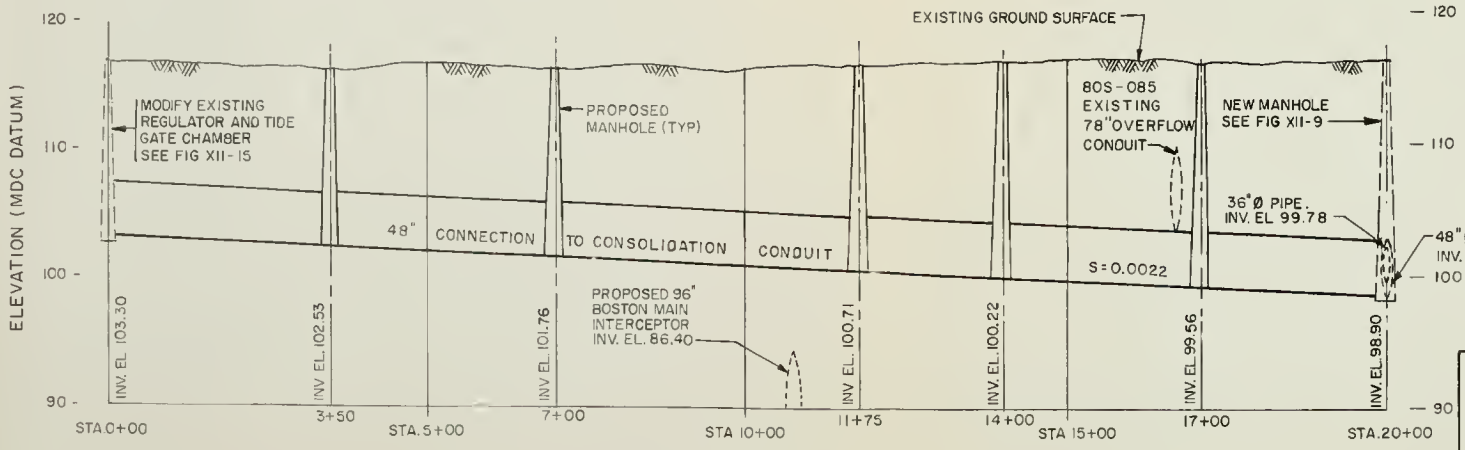
**FIG. XII-9**
**PROPOSED CONNECTION OF OUTFALL BOS-085 TO CONSOLIDATION CONDUIT**


MH STA 11+75
INTERCEPTOR
MERCER
ST.
DEVINE WAY
SOUTH BOSTON
EXISTING REGULATOR MANHOLE
EXISTING TIDE GATE (TO BE ABANDONED)
PROPOSED JUNCTION STRUCTURE SEE FIG. XII-9
EXISTING SOUTH BOSTON INTERCEPTOR
MH STA 7+00
MH STA. 3+50
NEW 48" CONNECTION TO CONSOLIDATION CONDUIT
MH STA 14+00
COLUMBIA ROAD
36" CONNECTION FROM BOS-085 SEE FIG. XII-9
MH STA 17+00
15" DRY WEATHER FLOW CONNECTION TO SOUTH BOSTON INTERCEPTOR
LOGAN WAY
OLD COLONY AVENUE
54"x57"
COLUMBIA ROAD
COLUMBUS PARK
APPROXIMATE LOCATION OF PROPOSED 96" BOSTON MAIN INTERCEPTOR (BY OTHERS - SEE TEXT)
MH STA. 20+00 SEE FIG. XII-9
124"x75"
OVERFLOW FROM SOUTH BOSTON INTERCEPTOR EXPECTED TO BE INACTIVE UPON COMPLETION OF NEW BOSTON MAIN INTERCEPTOR - SEE TEXT
GAVIN WAY
MODIFY EXISTING REGULATOR AND TIDE GATE CHAMBER SEE FIG. XII-15
BOS-085 OUTFALL (60"x60")
TO NEW CONSOLIDATION CONDUIT - SEE FIG. XII-9
BOS-086 OUTFALL (124"x75")

PLAN

200 0 200

SCALE IN FEET



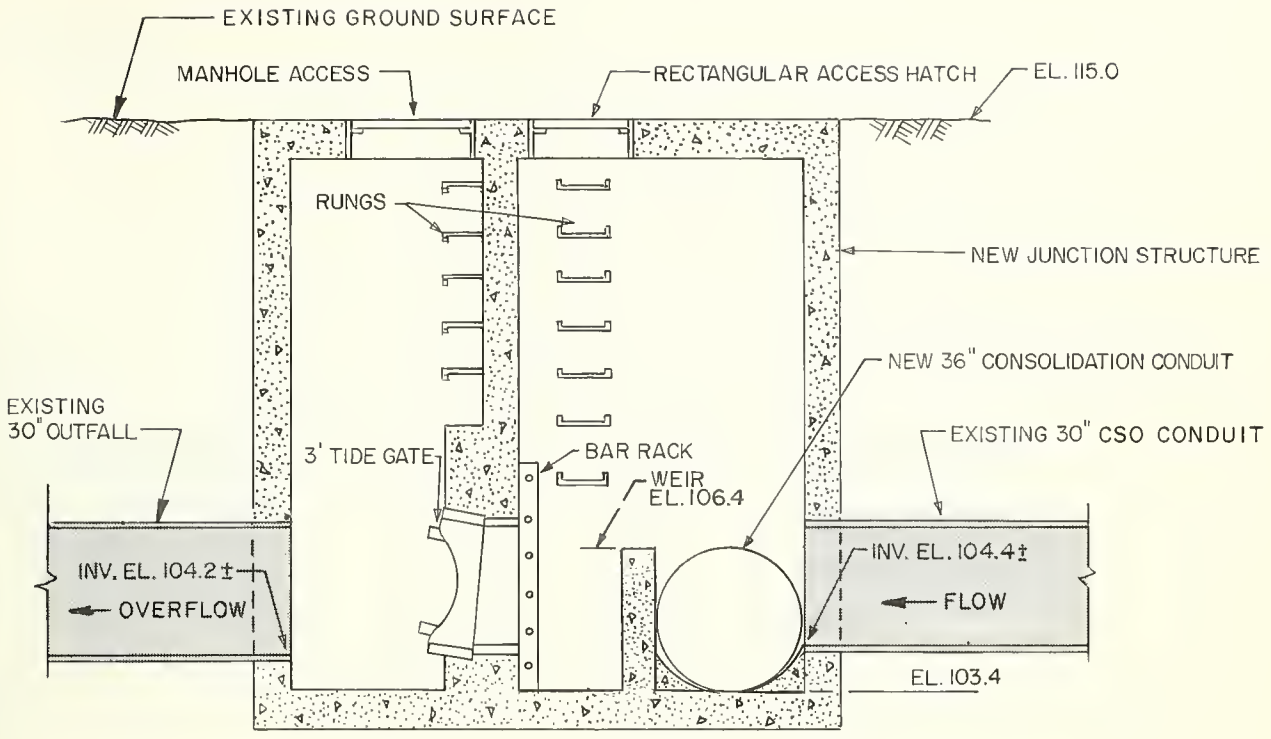

PROFILE

HORIZONTAL SCALE IN FEET 200 0 200

VERTICAL SCALE IN FEET 10 0 10

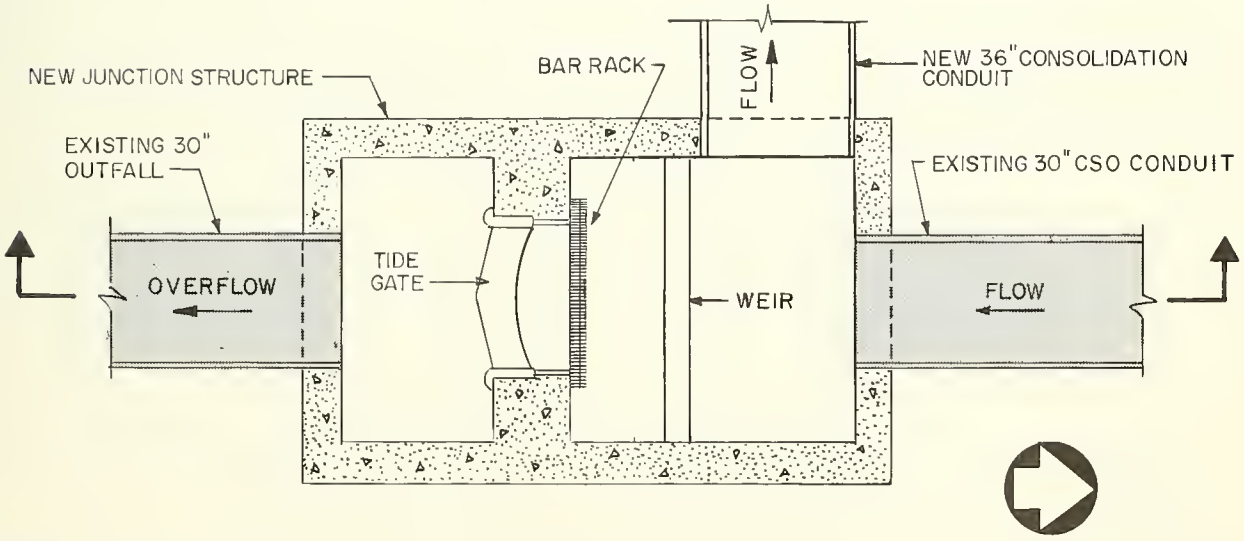
COMMONWEALTH OF MASSACHUSETTS
METROPOLITAN DISTRICT COMMISSION
COMBINED SEWER OVERFLOW PROJECT
DORCHESTER BAY AREA FACILITIES PLAN

PROPOSED
CONNECTION OF OUTFALL BOS-086
TO CONSOLIDATION CONDUIT
FIG. XII-10




SECTION

SCALE: 1/4" = 1'-0"




NOTE:
1. SEE FIG. XII-3 FOR LOCATION
2. ALL ELEVATIONS ARE M.D.C. DATUM

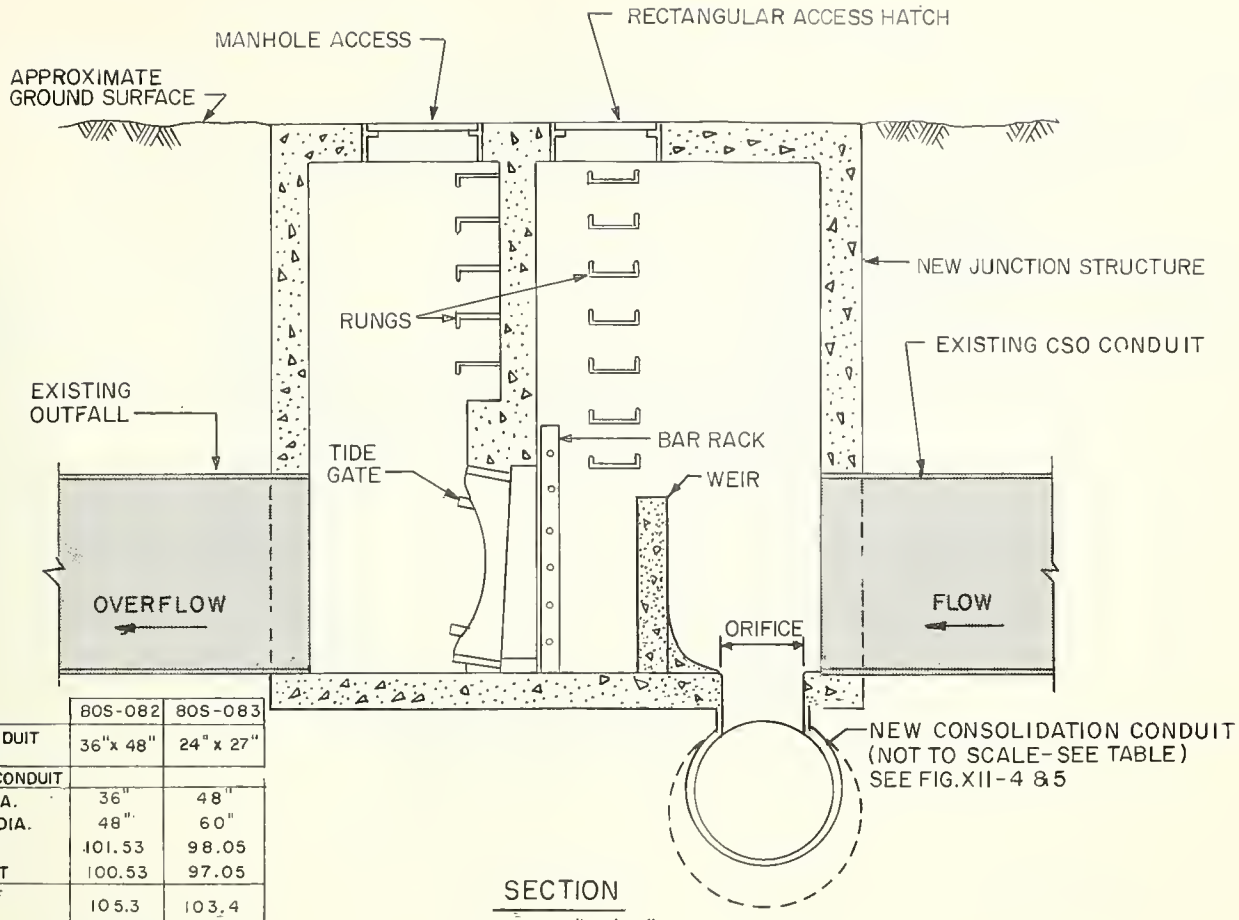
PLAN

SCALE: 1/4" = 1'-0"

▭: EXISTING

FIG. XII-11 CONSOLIDATION CONDUIT JUNCTION STRUCTURE AT OUTLET BOS 081

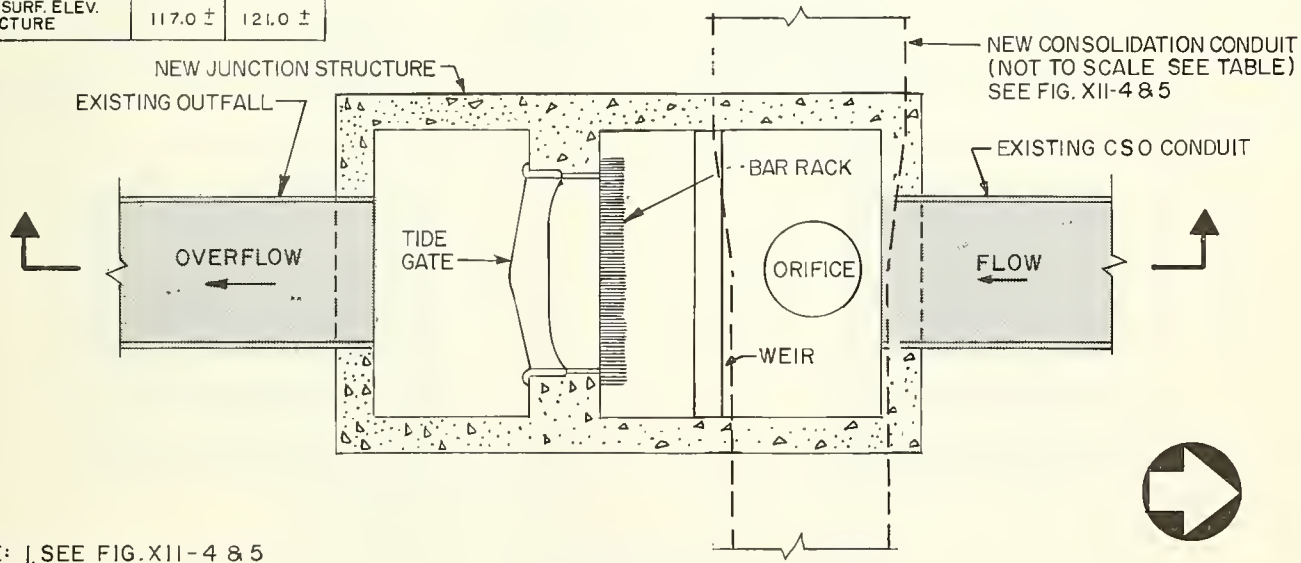
COMMONWEALTH OF MASSACHUSETTS
METROPOLITAN DISTRICT COMMISSION
COMBINED SEWER OVERFLOW PROJECT
DORCHESTER BAY AREA FACILITIES PLAN




| | 80S-082 | 80S-083 |
|---|---|---|
| EXIST. CSO CONDUIT SIZE | 36"x 48" | 24"x 27" |
| NEW CONSOLID CONDUIT | | |
| SIZE IN, DIA. | 36" | 48" |
| SIZE OUT, DIA. | 48" | 60" |
| INVERT IN | 101.53 | 98.05 |
| INVERT OUT | 100.53 | 97.05 |
| ELEV. INVERT OF STRUCTURE | 105.3 | 103.4 |
| WEIR CREST ELEV. | 109.0 | 109.4 |
| ORIFICE SIZE, DIA. | 24" | 10" |
| GROUND SURF. ELEV. AT STRUCTURE | 117.0 ± | 121.0 ± |

SECTION

SCALE: 1/4" = 1'-0"




NOTE: 1. SEE FIG. XII-4 & 5 FOR LOCATION
2. ALL ELEVATIONS ARE M.D.C. DATUM.

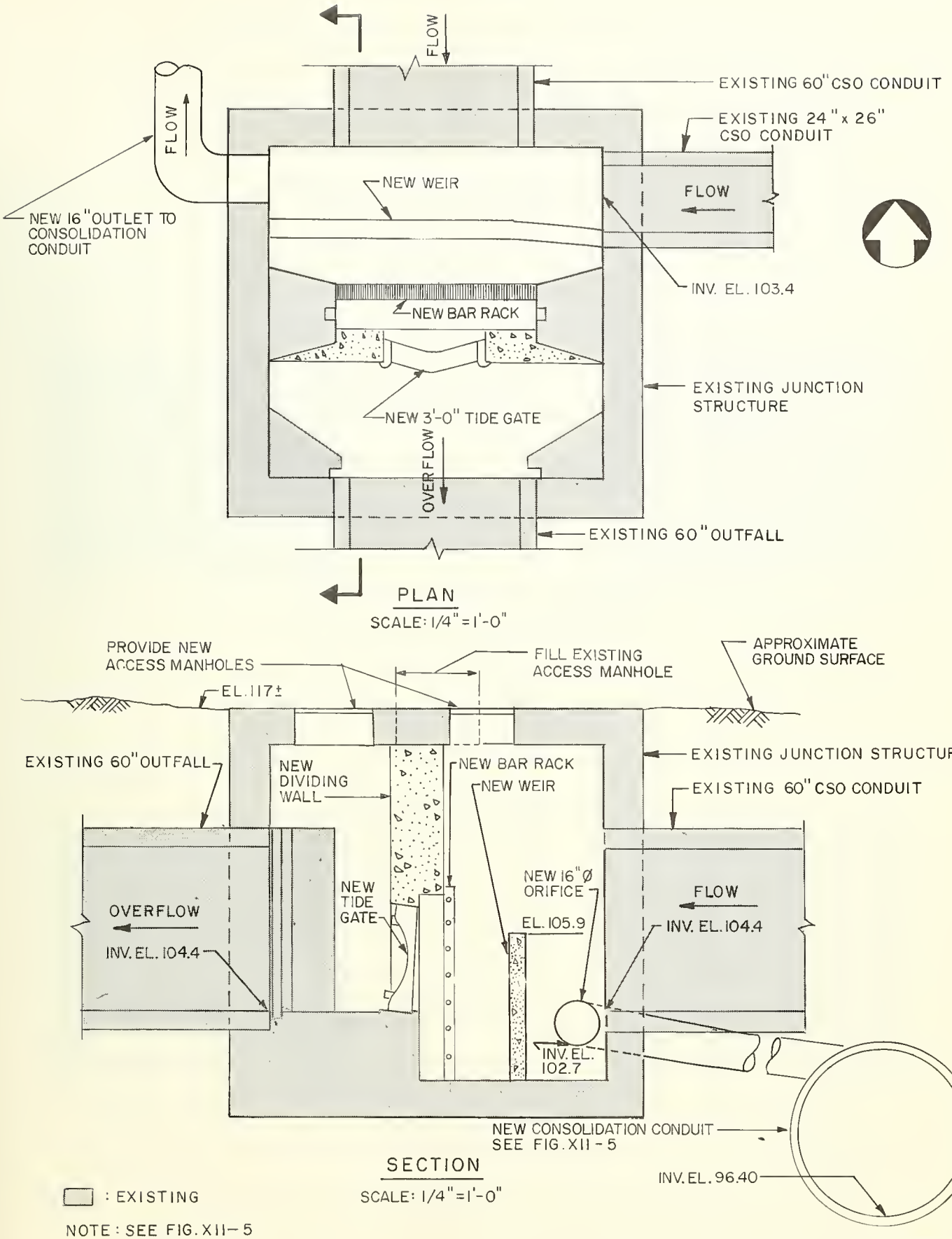
PLAN

SCALE: 1/4" = 1'-0"

▭ : EXISTING

# FIG. XII-12 CONSOLIDATION CONDUIT JUNCTION STRUCTURE AT OUTLETS BOS 082 & 083

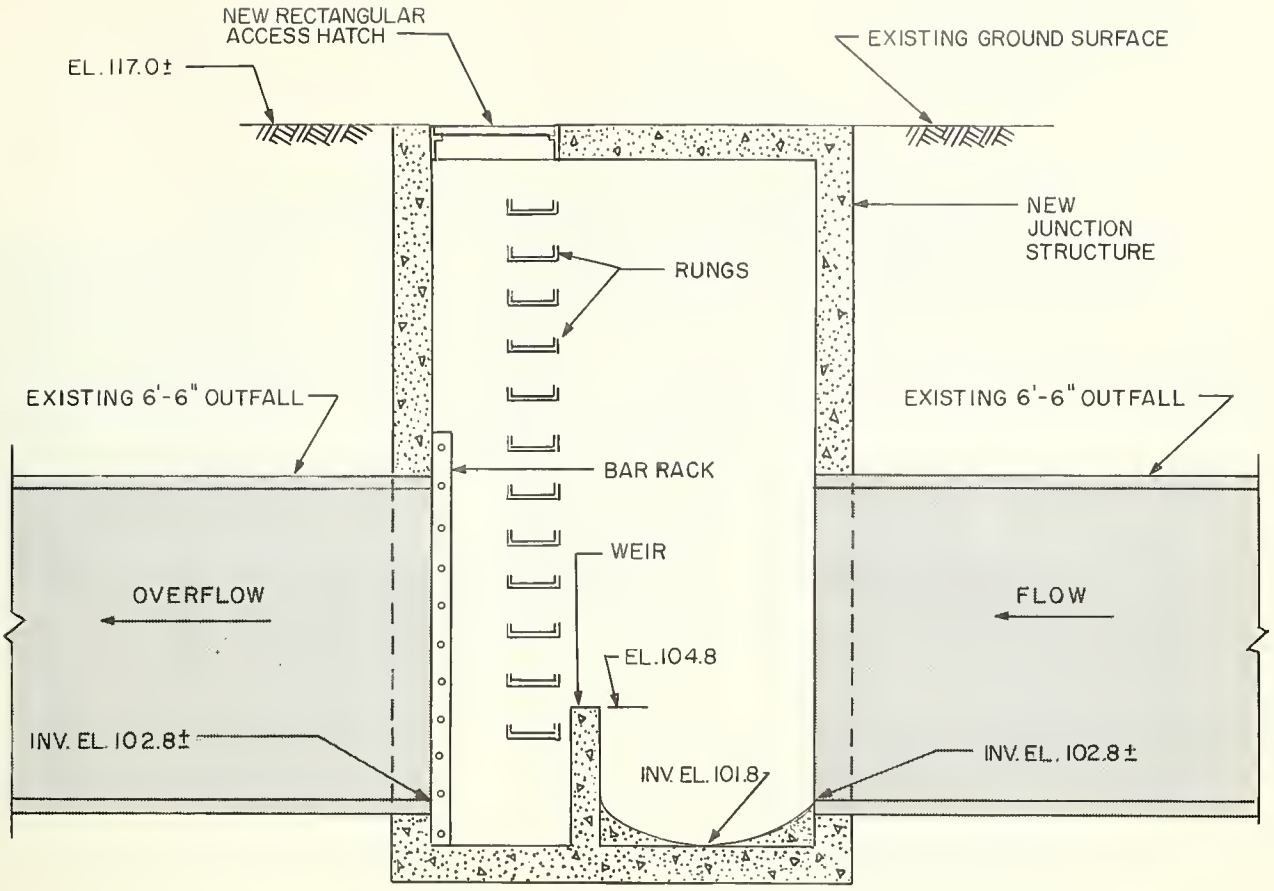
COMMONWEALTH OF MASSACHUSETTS
METROPOLITAN DISTRICT COMMISSION
COMBINED SEWER OVERFLOW PROJECT
DORCHESTER BAY AREA FACILITIES PLAN




# FIG. XII-13 CONSOLIDATION CONDUIT JUNCTION STRUCTURE AT OUTLET BOS 084

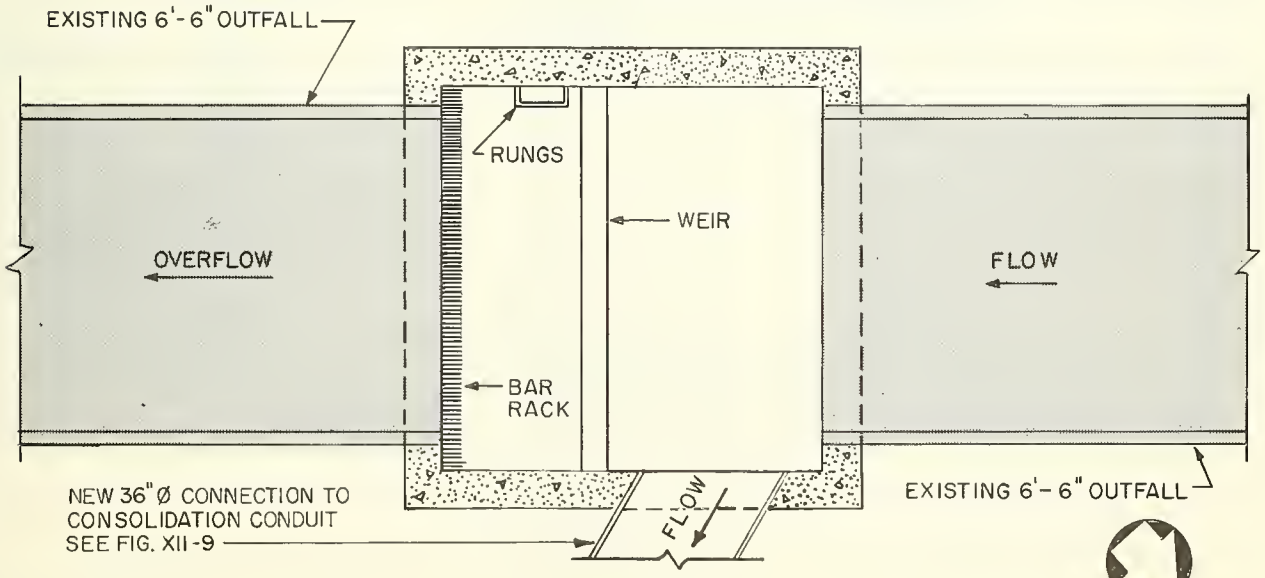
COMMONWEALTH OF MASSACHUSETTS
METROPOLITAN DISTRICT COMMISSION

COMBINED SEWER OVERFLOW PROJECT
DORCHESTER BAY AREA FACILITIES PLAN




SECTION

SCALE: 1/4" = 1'-0"

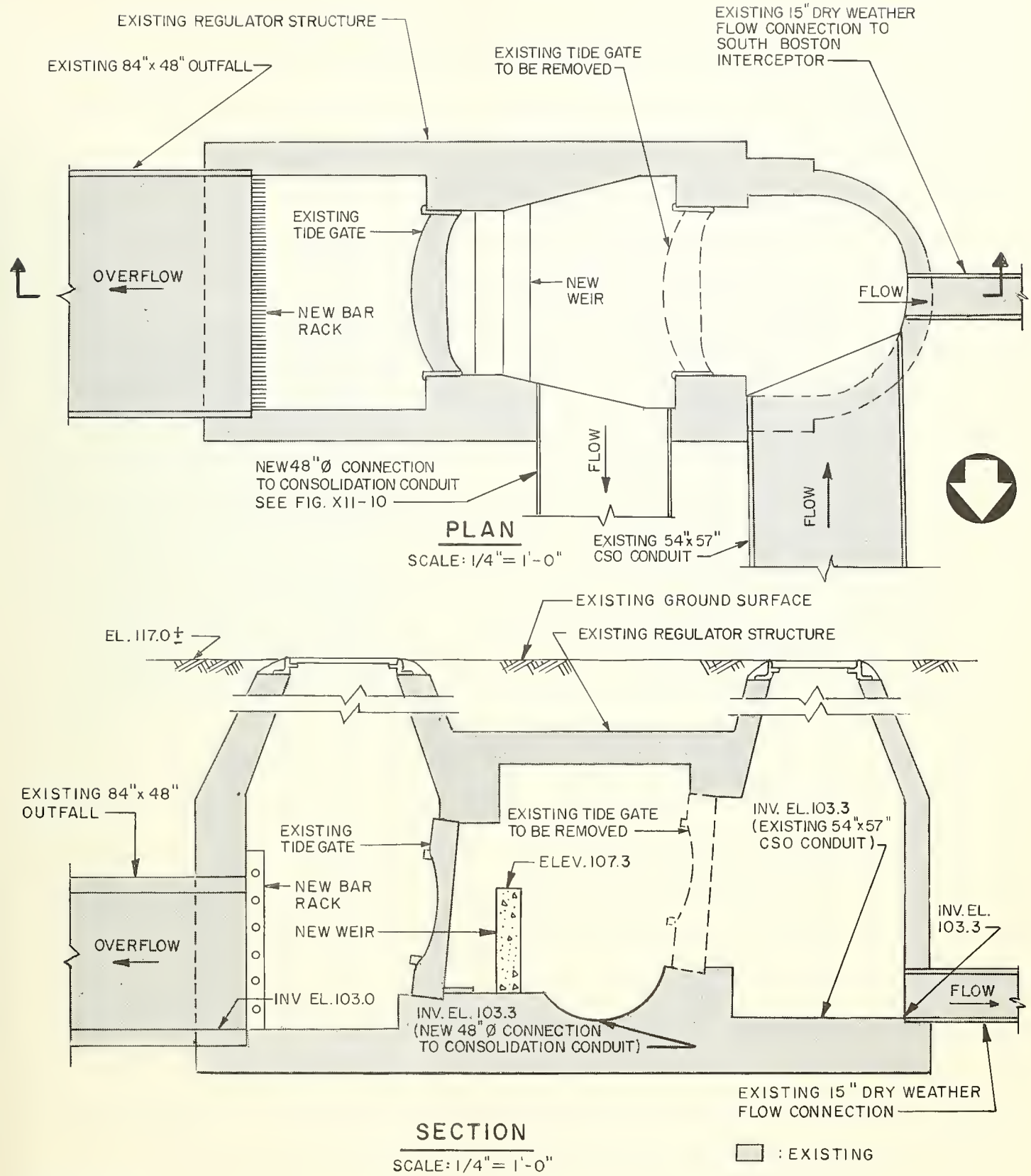



PLAN

SCALE: 1/4"=1'-0"

FIG. XII-14 BOS-085 JUNCTION STRUCTURE FOR CONNECTION TO CONSOLIDATION CONDUIT

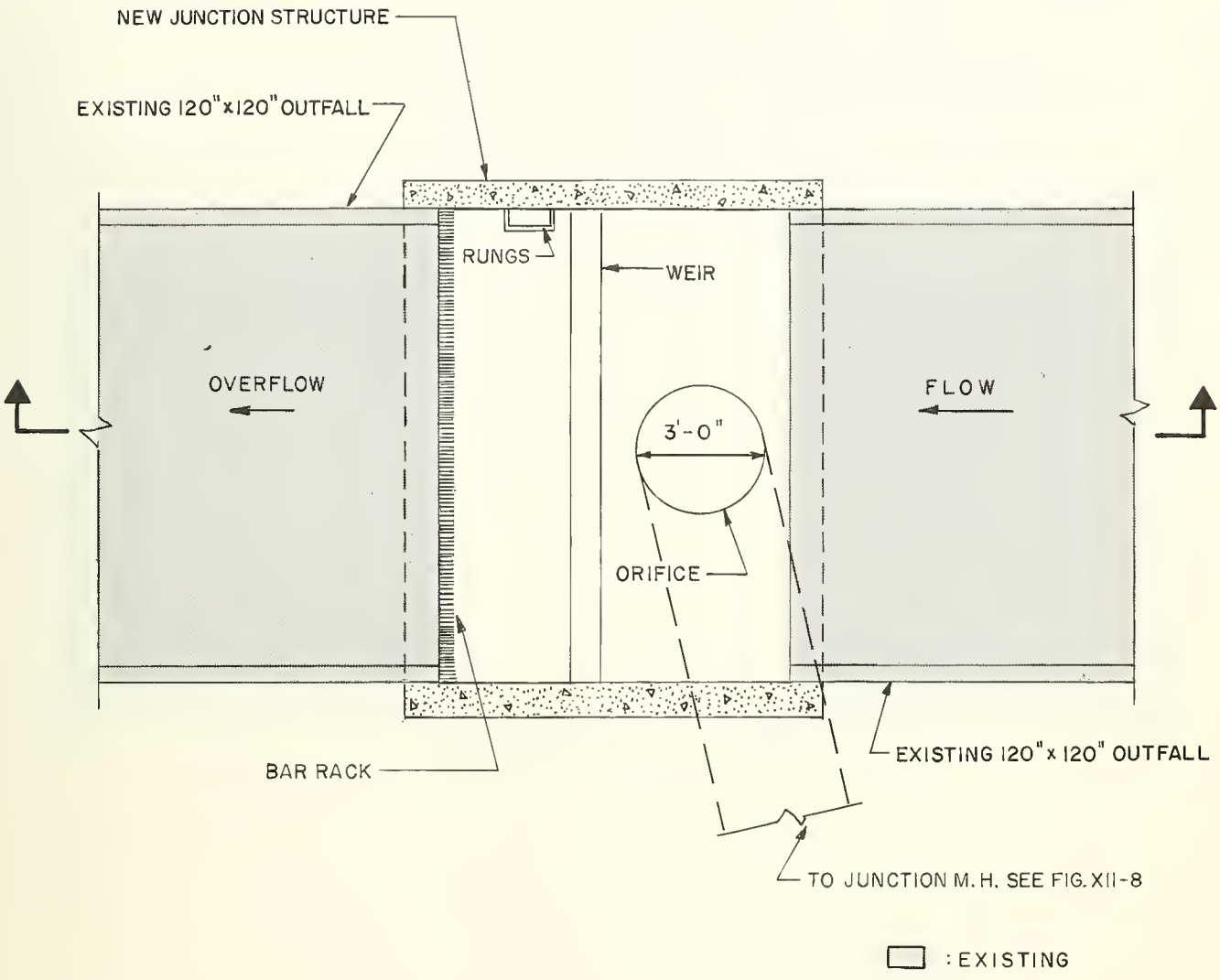
COMMONWEALTH OF MASSACHUSETTS
METROPOLITAN DISTRICT COMMISSION
COMBINED SEWER OVERFLOW PROJECT
DORCHESTER BAY AREA FACILITIES PLAN




NOTE: SEE FIG. XII-10 FOR LOCATION.

# FIG. XII-15 REGULATOR MODIFICATION FOR BOS-086 CONNECTION TO CONSOLIDATION CONDUIT

COMMONWEALTH OF MASSACHUSETTS
METROPOLITAN DISTRICT COMMISSION
COMBINED SEWER OVERFLOW PROJECT
DORCHESTER BAY AREA FACILITIES PLAN



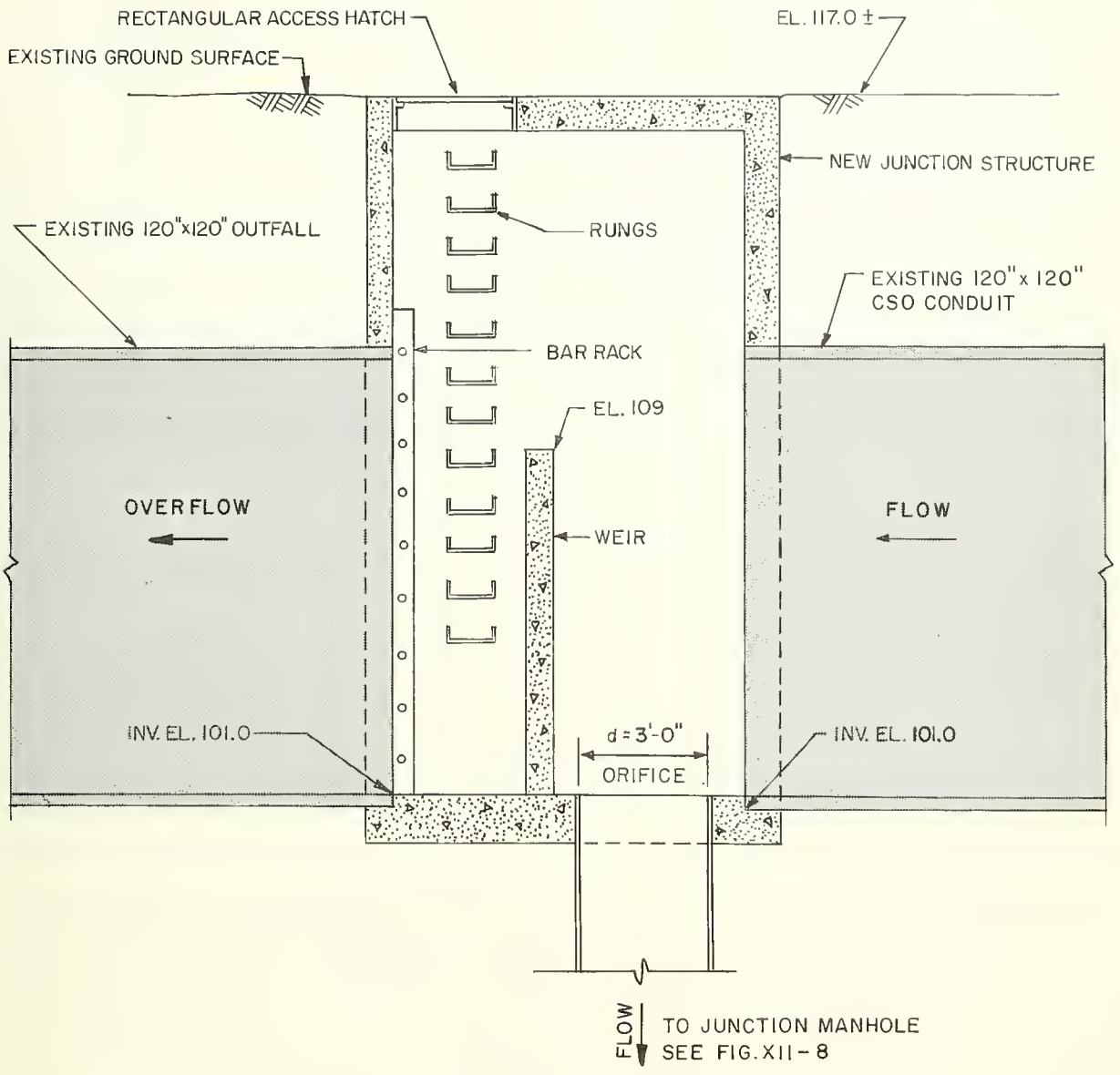

PLAN

SCALE: 1/4" = 1'-0"

NOTE:
SEE FIG. XII-8
FOR LOCATION

FIG. XII-16A PROPOSED JUNCTION STRUCTURE ON CONSOLIDATION CONDUIT AT BOS-087

COMMONWEALTH OF MASSACHUSETTS
METROPOLITAN DISTRICT COMMISSION
COMBINED SEWER OVERFLOW PROJECT
DORCHESTER BAY AREA FACILITIES PLAN




SECTION

SCALE: 1/4"=1'-0"

: EXISTING

NOTE:

SEE FIG. XII-8
FOR LOCATION

FIG. XII-16B PROPOSED JUNCTION STRUCTURE ON CONSOLIDATION CONDUIT AT BOS-087

COMMONWEALTH OF MASSACHUSETTS
METROPOLITAN DISTRICT COMMISSION
COMBINED SEWER OVERFLOW PROJECT
DORCHESTER BAY AREA FACILITIES PLAN

TABLE XII-1 SUMMARY OF DESIGN CRITERIA FOR CONSOLIDATION CONDUIT

| Consolidation Conduit Reach | Length, ft. | Tributary Area, Acres | Design Flow, mgd | Diameter, in. | Slope, ft/ft | Nominal Flow Capacity, mgd | Velocity at Des. Flow, fps |
|---|---|---|---|---|---|---|---|
| BOS-081 to BOS-082 (Sta. 0+0 to Sta. 18+70) | 1,870 | 27 | 12.4 | 36 | 0.001 | 13.5 | 2.6 |
| BOS-082 to BOS-083 (Sta. 18+70 to Sta. 36+40) | 1,770 | 99 | 30.7 | 48 | 0.002 | 37.0 | 4.5 |
| BOS-083 to BOS-084 (Sta. 36+40 to Sta. 47+70) | 1,130 | 128 | 34.7 | 60 | 0.0006 | 35.0 | 2.7 |
| BOS-084 to BOS-085/086 (Sta. 47+70 to Sta. 64+50) | 1,680 | 171 | 41.7 | 60 | 0.0007 | 40.0 | 3.1 |
| BOS-085/086 to BOS-086 Crossing (Sta. 64+50 to Sta. 72+30) | 780 | 353 | 86.8 | 60 | 0.003 | 85.0 | 6.3 |
| BOS-086 Crossing to Tank Junction Structure (Sta. 72+30 to Sta. 90+40) | 1,810 | 353 | 86.8 | 72 | 0.001 | 80.0 | 4.2 |
| BOS-087 Direct Conn. to Tank Junction Structure | 30 | 190 | 62.4 | 36 (orifice) | - | 65.0 | - |
| Tank Junction Structure to Tank Facility | 20 | 543 | 149.2 | 84 | 0.002 | 170.0 | 6.4 |
| Connection - BOS-086 Regulator to Jnct. with BOS-085 connection to Consol. Conduit (see Fig. XII-10) | 2,000 | 113 | 37.8 | 48 | 0.0022 | 38 | 4.6 |
| Connection - BOS-085 Regulator to Jnct. with BOS-086 Connection (see Fig. XII-9) | 500 | 47 | 26.2 | 36 | 0.0045 | 26.0 | 5.6 |
| Connection - Jnct. of BOS-085 & BOS-086 Connections to Consolid. Conduit at Sta. 64+50 (see Fig. XII-9) | 610 | 160 | 55.0 | 48 | 0.0045 | 55.0 | 6.2 |

outlets downstream of Day Boulevard, ambient water levels in the existing outfall conduits fluctuate with tide from about 1-ft. depth at low tide to completely filled at high tide. Accordingly, the control of CSO flowrates, to the proposed consolidation conduit at these locations would not be possible under most tidal conditions due to the flooding of the junction structures when the tidal level exceeded the elevation fo the diversion weir. If new tide gates were installed on each existing CSO conduit downstream of the junction structures, flowrate still could not be accurately controlled under tidal conditions, since a free-flow condition in the conduit would not exist. Under those conditions, flow would back up in the existing CSO conduit behind the tide gate and increase the head over the junction structure's outlet orifice which controls the rate of flow to the consolidation conduit. The increased head would result in a flowrate that would, most likely, exceed the design flowrate. A dynamic regulator, which would vary the orifice size as the head fluctuates, could be utilized to limit flowrate, however, static devices are preferred since they are more reliable and essentially maintenance free. It was, therefore, decided that CSO's from outlets BOS-085 and BOS-086 be conveyed to the consolidation conduit by new connecting conduits extending generally from the existing regulators, which are located upstream of existing tide gates (Figures XII-9 and XII-10). A diversion weir would be installed in the BOS-086 regulator as shown on Figure XII-15, and a new junction structure located just downstream of the BOS-085 regulator would be provided as shown on Figure XII-14. The use of connecting conduits proved to be more cost-effective than routing the consolidation conduit in Columbia Road to connect to the existing regulators.

The junction structures and regulator modifications required to divert CSO's from the existing system to the proposed consolidation conduit are shown on Figures XII-11 through XII-16. Each junction structure consists of a reinforced concrete chamber containing a diversion weir designed to divert CSO's up to the peak rate of flow from the 3-month, 4-hour design storm through a connection to the consolidation conduit. Junction structures for outlets BOS-081, 082, 083, and 084 include an adjacent downstream tide gate chamber, required to prevent tide water intrusion into the flow diversion chamber. The junction structure for outlets BOS-085 and BOS-087, and the regulator modification for outlet BOS-086, do not require new tide gates since existing tide gates are located downstream, within the CSO conduits. Each junction structure and the modified regulator would contain a stationary, manually cleaned bar rack downstream of the diversion weir which would serve to remove large floatables from CSO's that exceed the design flowrate. It is estimated that these instances would occur on an average of about 4 times per year. Each junction structure and modified regulator would not be significantly affected by downstream tide levels, thus assuring accurate flow regulation.

The size and slope of the proposed consolidation conduit was based on conveying design flowrates to the storage/containment facility while also minimizing the depth of excavation during construction. In addition, adequate cleansing velocities in the range of 2.5 to 7.0 fps also served as criteria in selecting pipe sizes and slopes. Since the major portion of the estimated construction cost of the conduit is for trench excavation, sheeting and dewatering, larger diameter pipe constructed at a relatively flat slope was selected. The depth of excavation at the storage/containment facility would also be minimized if the consolidation

conduit was constructed on a relatively flat slope. Design criteria including pipe sizes, slopes, design flow, and velocities for each reach of conduit is shown on Table XII-1.

Foundation Requirements and Construction Procedures. Preliminary foundation requirements and construction procedures were developed in conjunction with Haley and Aldrich Inc. (H&A) of Cambridge, Massachusetts, Consulting Geotechnical Engineers and Geologists. Test borings were drilled in Day Boulevard along the proposed route of the consolidation conduit to obtain information on existing subsurface soil and groundwater conditions. Subsurface information was also obtained from available historical sources. Based on the results of an analysis of this data, it is recommended that the conduit be supported for its entire length on naturally deposited inorganic soil or compacted structural fill placed after excavation of existing fill and organic soil. Construction procedures would require a braced, sheeted trench excavation with the steel sheeting extending, for the most part, about 10-ft. below the bottom of the trench. The report by Haley & Aldrich, which includes detailed discussions on subsurface explorations, subsurface conditions, preliminary design recommendations and construction considerations for the consolidation conduit is included in Appendix C.

Estimated Costs. The total estimated capital cost of the proposed consolidation conduit, including junction structures and regulator modifications, is summarized as follows (1979 Price Levels-ENR 2900):

| | |
|---|---|
| Sheeting, Excavation and Dewatering (incl. structures) | $6,410,000 |
| Pipe | 730,000 |
| Structures (incl. regulator modification) | 260,000 |
| Subtotal | 7,400,000 |
| Engineering and Contingencies (30%) | 2,220,000 |
| Total | $9,620,000 |

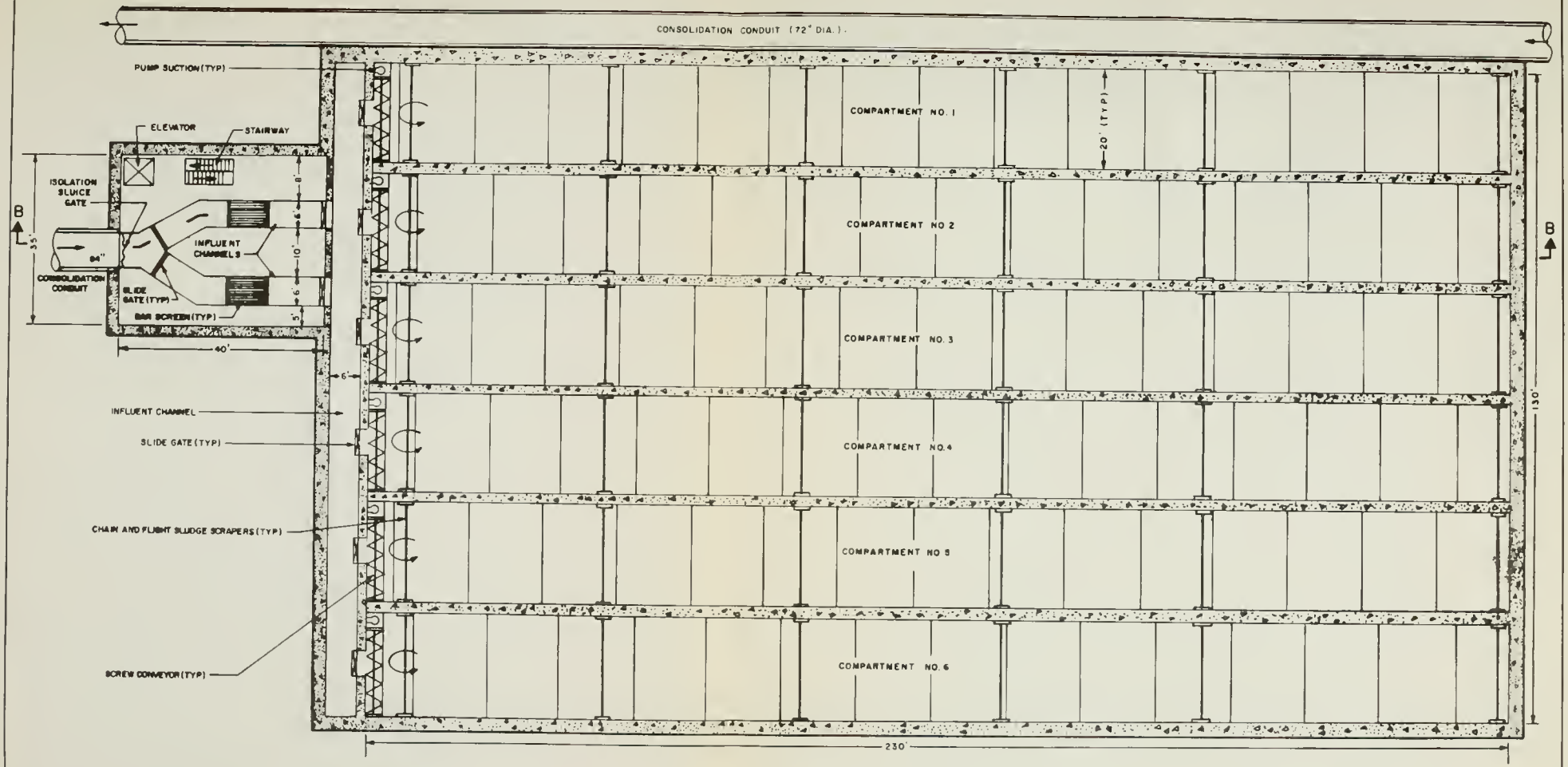
Storage/Containment Facility. As discussed earlier in this chapter, the effective off-line storage volume required to contain CSO's generated from the 3-month, 4-hour design storm is 3.0 mil. gal. As shown previously on Figure XII-2, the selected site of the storage/containment facility is located east of the existing MDC Police Station (Old Colony Division) between Day Boulevard and the Boston Teachers Union parking lot. A detailed description of the facilities follows.

Summary of Design Criteria. A summary of the design criteria for the South Boston storage/containment facility used for preliminary design, is presented in Table XII-2.

General Description. Figures XII-17, 18, and 19 show a plan view, sectional view and details, respectively, of the recommended facilities. As shown, flow from the consolidation conduit would enter the channel room (invert elevation - 91.17, MDC datum) and flow through one of two 6-ft. wide x 8-ft. deep influent channels, each containing a mechanically, cleaned bar screen. Two channels are proposed in the event one of the bar screens is out of service for maintenance or repair reasons.

TABLE XII-2 SUMMARY OF DESIGN CRITERIA FOR THE SOUTH BOSTON STORAGE/CONTAINMENT FACILITY

| | |
|---|---|
| Tributary Area, acres | 543 |
| Design Storm | 3-mo, 4-hr |
| Design Flowrate, mgd | 150 |
| Design Volume, mil. gal. | 3.0 |
| Coarse Screen Bar Spacing, in. | 1.5 |
| Type of screen adopted for preliminary design | catenary |
| Estimated volume of screenings: | |
| from design storm, cu.ft. | 10 |
| average per year, cu.ft. | 260 |
| Number of tank compartments | 6 |
| Volume per compartment - effective mil. gal. | 0.50 |
| nominal, mil. gal. | 0.72 |
| Total Volume - effective, mil. gal. | 3.00 |
| nominal, mil. gal. | 4.33 |
| Sludge Collection | Chain and flight scrapers and screw conveyors |
| Pumpback - no. of pumps | 3 (1 standby) |
| capacity per pump, mgd (gpm) | 3.0 (2,080) |
| TDH per pump, ft. | 18 |
| force main diameter, in. | 18 |
| time to drain tank at design storm, hr. | 12 |
| Estimated total annual waste stream volume through facility, mil. gal. | 130 |
| Estimated annual total suspended solids load through facility, lbs. | 55,000 |
| Estimated annual volume of grit through facility, cu.ft. | 800 |




PLAN AT ELEV. 91.9

SCALE: 1" = 20'

20 0 20 40

SCALE IN FEET

COMMONWEALTH OF MASSACHUSETTS

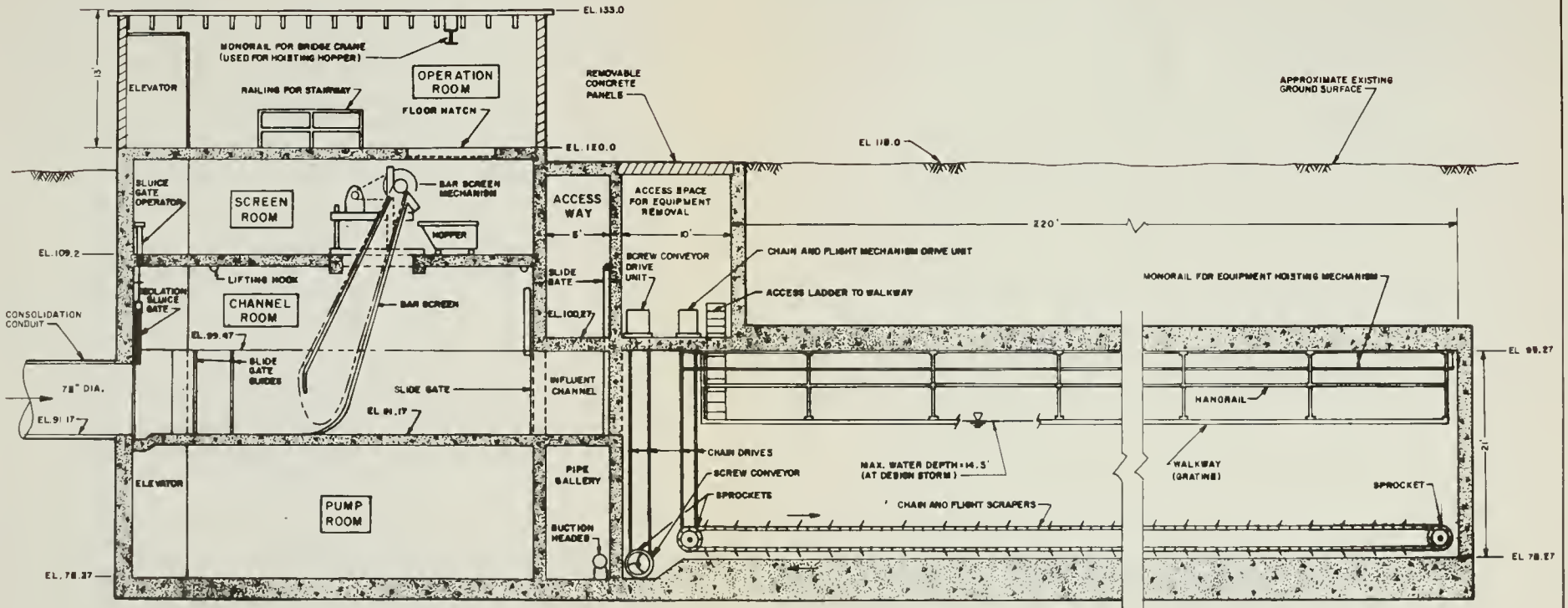
METROPOLITAN DISTRICT COMMISSION

COMBINED SEWER OVERFLOW PROJECT

DORCHESTER BAY AREA FACILITIES PLAN

SOUTH BOSTON STORAGE/CONTAINMENT FACILITY(PLAN)

FIG. XII – 17




SECTION B-B

SCALE: 1" = 10'

NOTES:

1. ALL ELEVATIONS REFER TO MDC DATUM
2. THIS SECTION REPRESENTS A PRELIMINARY LAYOUT; VALVES AND APPURTENANCES NOT SHOWN

10 0 10 20

SCALE IN FEET

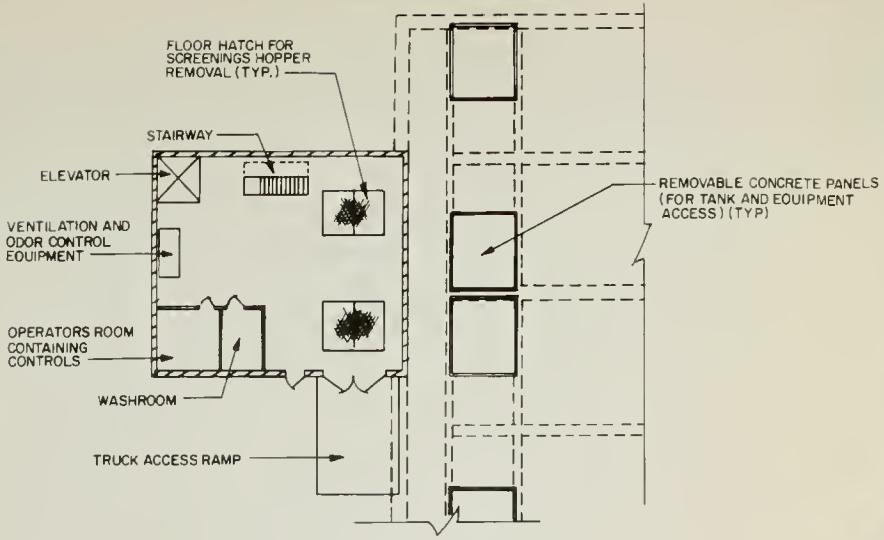
COMMONWEALTH OF MASSACHUSETTS

METROPOLITAN DISTRICT COMMISSION

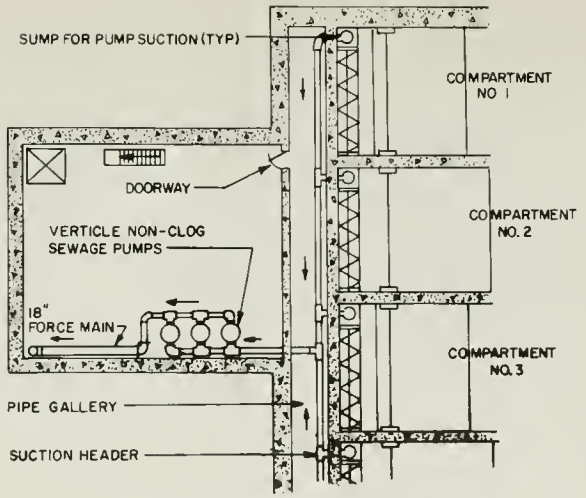
COMBINED SEWER OVERFLOW PROJECT
DORCHESTER BAY AREA FACILITIES PLAN

SOUTH BOSTON STORAGE/CONTAINMENT FACILITY (SECTION)

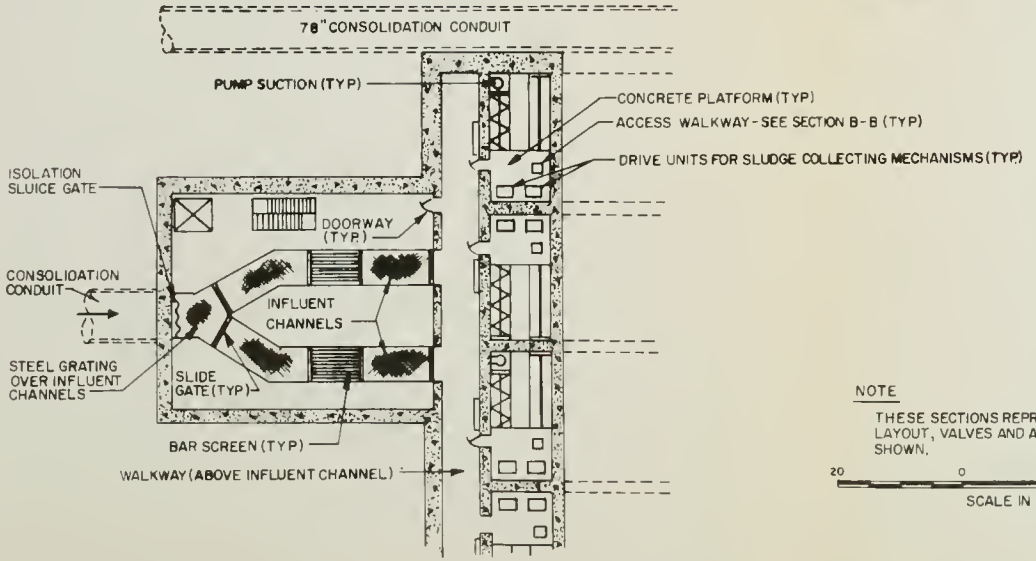
FIG. XII-18




PARTIAL PLAN AT ELEV. 120.0




PARTIAL PLAN AT ELEV. 78.27




PARTIAL PLAN AT ELEV. 101.0

NOTE

THESE SECTIONS REPRESENT A PRELIMINARY LAYOUT, VALVES AND APPURTENANCES NOT SHOWN.

20 0 20 40

SCALE IN FEET

COMMONWEALTH OF MASSACHUSETTS

METROPOLITAN DISTRICT COMMISSION

COMBINED SEWER OVERFLOW PROJECT

DORCHESTER BAY AREA FACILITIES PLAN

SOUTH BOSTON STORAGE/CONTAINMENT FACILITY (DETAILS)

FIG. XII-19

Manually operated slide gates installed in each channel (Figure XII-17) would direct flow through the selected channel. Each bar screen would consist of a mechanically cleaned, catenary-type unit with the drive unit and screenings hopper located on the above floor level (elevation 109.2). The recommended bar spacing is 1.5-in. which would serve to remove large solids and protect the pumping units which would pump the stored volume to the existing sewerage system. This type of screening mechanism was selected based on the results of an analysis performed during the preliminary design of the Dorchester screening/disinfection facilities on three types of screens. A summary of this analysis, including a description of the catenary-type screening unit, is presented later in this chapter, in the section discussing the screening/disinfection facilities.

The screenings would be discharged from the rakes into a storage hopper located on the above floor level. The estimated volume of screenings generated by the design storm is about 10 cu. ft., and the total annual volume of screenings is estimated to be about 260 cu. ft. A standard size 1 cu. yd. (27 cu. ft.) capacity hopper would be provided for each bar screen. A monorail and bridge crane system located in the superstructure would be used to hoist the hoppers vertically through a floor hatch to the superstructure floor (elevation 120.0), and horizontally through doors to the truck access ramp (Figure XII-19). The hoppers would be emptied into a dump truck outside of the building to avoid operating the vehicle inside of the structure where potentially explosive gases may be present. The ultimate disposal of the screenings will be discussed later in this chapter.

After the waste stream has passed through the screening unit it would enter the influent channel for discharge into any one of the six tank compartments, each having an effective volume of about 500,000 gal. Flow between and within the tank compartments would occur through a rectangular opening in the side wall which would act as a side overflow weir. The bottom elevation of the openings would be set at elevation 92.3 which is 0.5 ft below the maximum effective depth in the tank (14.5 ft.).

Under normal operation, the slide gates controlling flow to compartments 2 through 6 would be closed, and the slide gate for compartment 1 open. Flow from the influent channel would enter compartment 1, and eventually overflow to compartment 2 if the CSO volume exceeds 500,000 gallons. As the volume increases, flow would enter the remaining compartments in series until the maximum water depth of 14.5-ft. is reached in all six compartments. If flow continues (i.e., if the storm CSO volumes exceed the design storm CSO volumes), the isolation sluice gate located in the channel room would automatically close, surcharging the consolidation conduit and causing flow to discharge to the receiving water through the existing CSO outfalls. This flow scheme could also be reversed by allowing flow to enter compartment 6 and flow in series to compartment 1. Both schemes would be practiced to equalize the use of sludge removal equipment.

As shown on Figures XII-17 and 18, sludge removal equipment for each compartment includes chain and flight scrapers which serve to transport settled solids into a trough located at the inlet end of the tank where a screw conveyor would convey them to a sump. As an alternative to this mechanical sludge removal method, the cost effectiveness and applicability of a diffused air system for keeping the solids in suspensions was investigated. According to equipment manufacturers, a stainless steel fixed header system with air diffusers would be required in each compartment with air generated from a centrally located blower. The air requirement is estimated to be about 25 cubic feet per minute (CFM) per 1000 cubic feet of tank volume, resulting in a total air requirement of about 1700 CFM per compartment (0.5 mil. gal.) or about 10,000 CFM for the entire tank (3.0 mil. gal.). This would require two to three blowers, each equipped with motors having horsepowers in the range of 100 to 200. The estimated capital cost of this system would be about $600,000. Considering the nature of the waste stream, which would contain grit and other readily settleable solids, the effectiveness of a diffused air system for keeping all solids in suspension is questionable. Due to the continually varying depth of flow within each compartment, the air requirements would continually vary during tank filling and also during draining. Accordingly, accurate control of air volumes required for effective solids suspension would not be possible and, most likely, a portion of the solids would settle on the bottom of the tank. The mechanical sludge removal system proposed, has a slightly higher present day estimated capital cost ($650,000), however, the drive units require only 3 to 5 horsepower motors. In addition to being significantly less energy intensive, the mechanical system offers a more positive means of sludge collection, thus assuring effective removal under all water level conditions. Accordingly, the mechanical sludge removal system has been adopted for preliminary design.

As shown on Figures XII-17, 18 and 19, the sludge collection mechanism drive units would be located on platforms above the inlet end of each compartment. Access to these units would be through the walkway located above the influent channel (see Figure XII-19, Partial Plan at Elev. 101.0). Each platform would also contain an access ladder leading to a walkway installed along a side wall of each tank compartment. This walkway would be used for visual inspection of the sludge removal equipment. A monorail equipped with a hoisting mechanism would be installed along the ceiling of each tank compartment and would be utilized for removing sludge removal equipment in the event major maintenance, repair work or replacement is required. Access to the tank for removal of the equipment would be provided at the inlet end of each compartment through removable concrete panels (Figures XII-18 and XII-19).

The stored CSO volume within the tank, including solids, would be pumped to the Columbus Park connection sewer just before it enters the Columbus Park headworks. As shown on Figure XII-18 and XII-19, a pump suction header would be installed in the pipe gallery along the inlet end of the tank. Pump suction for each compartment would be located in the sump which accepts the solids from the screw conveyor. Three vertical non-clog sewage pumps located in the pump room would serve to convey the

stored volume to the headworks through an 18-in. diameter force main. Each pump would be rated at 3 mgd (2,080 gpm) at a 18-ft. total dynamic head (TDH). These size pumping units were selected to provide a maximum pumpback rate of about 6 mgd with two pumps operating (one pump would serve as a standby unit), thus draining the maximum tank volume of 3.0 mil. gal. over a 12-hour period. The 18-in. diameter force main would be about 3,000-ft. long, extending from the storage facility, northward in Day Boulevard to the Columbus Park connection, approximately 100-ft upstream of the headworks. The force main would be installed in the consolidation conduit trench at an elevation that would provide about 6-ft. of cover over the pipe.

As presently conceived, the pumpback system would be primarily controlled by a signal from the headworks, which would allow pumpback to occur when excess flow capacity became available. A liquid depth sensing system (i.e., air bubbler pressure differential, etc.) would be installed in the tank to control the "on-off" sequencing of the pumps. Under most storm events, flow to the headworks from the Columbus Park connection and from the new Boston Main interceptor would be below the headwork's capacity of 180 mgd, thus pumpback could initiate as soon as the water level in the tank reaches a pre-set pump on control level. For large storm events when the headwork's capacity is reached, the pumpback system would initiate with one 3 mgd capacity pump when the flowrate reduces to about 177 mgd, and with two 3 mgd capacity pumps (total = 6 mgd) when the flowrate at the headworks reduces to about 174 mgd.

Superstructure. As presently conceived, the superstructure would consist of a 40-ft. x 35-ft x 13-ft. high building with brick and concrete masonry walls and a flat roof of prestressed concrete double-tee beam construction. The proposed monorail and bridge crane would be suspended from separate steel beams. As shown on Figure XII-19, the operator's room and washroom would be partitioned off within the main superstructure to reduce heating requirements. Floor hatches would be installed over the screening hoppers which are located on the level below, to allow the hoppers to be hoisted by the bridge crane during the screenings removal operation. An elevator and stairway would also be provided for access to lower levels.

Odor control equipment has been included in the preliminary layout and cost estimate of the storage/containment facility. As presently conceived, an air exchange unit would exhaust air from the entire structure incuding the tank, pass it through activated carbon contained in plastic cells, and vent it through the roof of the structure. Fresh air would be drawn into the tank compartments and operation rooms through vents. A sill cock and hose would also be provided for washing down the screens between operations in order to reduce odors from accumulations on the screens.

It is recommended that during final design, space for odor control equipment should be provided, but the actual need for odor control should be determined after the facility has been in operation for a period of time. The Columbus Park headworks for example, installed an ozone system after the facility had been in operation for a period of time for controlling strong odors emanating from the waste stream. Recently, however, use of

the ozone equipment has not been required since the severity of waste stream odor has significantly lessened. Conversely, the recently completed Union Park Street pumping station is experiencing severe odors and is using a hypochlorite spray system for control. Because of the nature of odor problems, operating experiencing is often required to determine if there is a need for control equipment and of what type.

Foundation Requirements and Construction Procedures. Similar to the consolidation conduit, preliminary foundation requirements and construction procedures for the storage/containment facility were developed in conjunction with Haley and Aldrich, Inc. Test borings were drilled near the final selected site (Figure XII-2) to obtain information on existing subsurface soil and groundwater conditions. A groundwater observation well was also installed at one of the boring locations. Based on the results of an analysis of this data, it is recommended that the structure be directly soil supported. Preliminary design indicates that a 4-ft. thick reinforced concrete mat is required for the base of the tank structure, supported on a working mat of granular fill overlain by 6 to 12 inches of lean concrete fill. Since the structure would extend below groundwater, it is recommended that it be designed to resist hydrostatic uplift pressures.

Construction of the facility would involve special considerations primarily relating to excavation and dewatering procedures due to the required depth of cut and the presence of clay in the substrata. Accordingly, three alternative methods for the proposed construction incorporating items relating to excavation, dewatering, lateral support and other pertinent geotechnical factors were considered during the preliminary design phase, primarily for cost estimating purposes. Construction scheme (A), which generally involves pre-stripping of the site, installation of steel sheet piling and staged excavation with appropriate internal cross bracing, proved to be the most cost-effective and was used in the development of the estimated construction cost of the facility. The report by Haley and Aldrich, which is included in Appendix C, presents a detailed discussion of all construction schemes considered, including illustrations. It also includes discussions on subsurface explorations (including boring logs), subsurface conditions and preliminary design recommendations.

Operation and Maintenance Requirements. The storage/containment facility would be manned during a storm event to assure proper operation of all equipment. Most of the functions associated with the operation of the storage/containment facility would be controlled automatically with manual override. The bar screens are self cleaning with the cleaning mechanism operating on a differential head sensor. The sludge scraping mechanisms could either be automatically controlled by a water level sensor in each compartment or manually controlled. The automatic control of the pumpback operation was described earlier in this chapter.

The depth and rate of flow in the main influent channel would be telemetered to the Columbus Park headworks in order to monitor the status of the facility and to determine when facility is activated. Upon completion of the pumpback operation, the screens would be washed down, all tank compartments and equipment would be inspected and serviced as required, the screenings hopper would be emptied into a truck for ultimate disposal, and the building secured. Daily inspection of facility between storm events would also be performed and equipment serviced and maintained as required. Estimated labor requirements are summarized as follows:

| Category | Civil Service Grade | Man-hours per week | Man-hours per year |
|---|---|---|---|
| Supervisory | 25-22 | 8.0 | 416.0 |
| Skilled Trades | 16-13 | 20.0 | 1040.0 |
| Unskilled | 9-6 | 20.0 | 1040.0 |
| Totals | | 48.0 | 2496.0 |

Energy requirements include the electricity to operate the bar screens, sludge collection equipment, pumps, bridge cranes, air compressors for the liquid depth air bubbler sensing systems, heating, ventilation and odor control equipment, lighting and other minor electrical requirements. It is estimated that approximately 80,000 kilowatt-hours of electricity per year would be utilized at the facility.

Removal of screenings would occur after each operation of the facility. For the purposes of this report it has been assumed to occur on the average of once per week. Most likely one MDC-owned light dump truck, similar to the one employed for screenings removal at the Columbus Park headworks, would be used to haul the screenings from the two Dorchester screening/disinfection facilities and the South Boston storage/containment facility to the Deer Island wastewater treatment plant for utlimate disposal. Miscellaneous equipment repair and maintenance would also be performed, as required, at the facility. A complete summary of the estimated annual costs associated with the operation and maintenance requirements is presented in the following sections.

Estimated Costs. Estimated capital costs and estimated annual operation and maintenance costs for the facility are summarized in Table XII-3. All costs represent 1979 price levels (ENR=2900).

TABLE XII-3 SUMMARY OF ESTIMATED COSTS FOR THE SOUTH BOSTON STORAGE/CONTAINMENT FACILITY

| ESTIMATED CAPITAL COSTS* | |
|---|---|
| Sheeting | $ 975,000 |
| Excavation | 1,770,000 |
| Structures | 9,520,000 |
| Mechanical Equipment and Controls | 1,870,000 |
| SUBTOTAL | $14,135,000 |
| Engineering and Contingencies (30%) | 4,245,000 |
| TOTAL | $18,380,000 |

| ESTIMATED ANNUAL OPERATION AND MAINTENANCE COSTS* | |
|---|---|
| Labor | $ 18,000 |
| Power | 13,000 |
| Screenings Removal | 2,000 |
| Miscellaneous Equipment Repair and Maintenance | 5,000 |
| TOTAL | $ 38,000 |

*Estimated costs reflect June 1979 price levels (ENR=2900)

CSO Outlet BOS-080. Four South Boston CSO outlets currently discharge into Reserved Channel: BOS-076, 078, 079 in the Inner Harbor Study Area, and BOS-080 in the Dorchester Bay Study Area. The Inner Harbor consultant performed a cost effective analysis to determine the most feasible method of treating the overflows. Results concluded that a centralized approach of treating BOS-076, 078, and 079 is warranted, however, BOS-080 should remain as part of the Dorchester Bay Study Area. Accordingly, as part of the recommended plan, the level of control and subsequent recommendations regarding BOS-080 were evaluated.

BOS-080 serves a 47 acre mixed land use, higher urbanized area of South Boston, and discharges into Reserved Channel at Farragut Road, extended. The tributary area is bordered by Independent Square on the west, Marine Park on the east, Fourth Street on the south, and First Street on the north. Overflows occur approximately 56 times per year, and the average annual overflow volume is 16.2 mil. gal. (from SEMSTORM for the 29-year period of record).

The water quality problem in the Reserved Channel is primarily floatables, oil and grease. The channel is used as a ship loading area serving the Castle Island and other terminals off of Boston Harbor. On an annual basis, the following pollutants discharge into Reserved Channel from BOS-080 based on SEMSTORM modeling:

| Pollutant Parameter | Estimated Annual Amount |
|---|---|
| Total Suspended Solids | 8,000 lb |
| $BOD_5$ | 6,700 lb |
| Total Coliform | $1.6 \times 10^{15}$ (no. per year) |

The above amounts show that the annual loads and subsequent impacts from BOS-080 CSO's are minor when compared to the total pollutant loads entering Boston Harbor. Also, the Reserved Channel (or receiving water) uses are primarily conducive to the discharge of pollutants, (i.e., ship spillage of fuel and waste materials, etc.) although such practices are not allowed. Considering the above, the necessity to provide any form of treatment facilities for the above pollutant parameters is questionable. Therefore, it has been assumed that the maximum treatment level, if any, would be the removal of floatables. In addition, the recommended improvements for BOS-080 should be assigned a low study area implementation priority.

Based on the premise that screening facilities, if anything, would be installed to treat the CSO's, the following three alternatives were considered:

1. Bar Screen in Existing Tide Gate Chamber - Under this alternative a manually cleaned bar screen would be installed in the existing tide gate structure located at the intersection of East First Street and Farragut Road (Figure B, appended). The major disadvantage with this alternative is its maintenance with regards to inaccessibility, since the access chamber is located in the middle of the intersection and the bottom of the stationary bar rack would be 15 feet below street grade. Further, and of greater consequence, the installation of a bar rack in the tide gate chamber could present a serious drainage obstruction in the event of a severe rainfall event coincident with an accumulation of floatables in the conduit. The estimated cost of this alternative is $5,000 (ENR 2900).

2. Screening Facility on Existing Overflow Conduit - This alternative consists of the installation of a mechanically cleaned screen on the existing 72" x 72" overflow conduit between East First Street and the Reserved Channel, with the headframe, drive mechanism and controls enclosed in an above-ground structure. However, examination of the land availability along the conduit route indicates that there is no available site for the installation, except at the discharge point in the Reserved Channel. Construction at this location would require a pile-supported platform extending out into the channel which would interface with normal shipping operations. In addition, access to the facility would be difficult in view of the present and proposed uses of the adjacent White Fuel Company and Massport parcels. White Fuel Company, which owns the land located north of East First Street and west of the CSO conduit (Figure B, appended), operates a high-volume operation which includes tanker unloading, fuel storage and tank-truck loading. Massport, which owns the property located north of Marine Park and east of the conduit, is in the process of constructing a marine-shipping container terminal facility at this site. If the facility could be located at the discharge point in the Reserved Channel, the estimated capital cost would be in the order of $2.0 million (ENR 2900).

3. Screening Facility on a Diversion Conduit - This alternative is similar in nature to the Dorchester screening/disinfection facilities (discussed later in this chapter) in that it would involve construction of a screening facility over a diversion channel. The channel would convey CSO's from the existing CSO conduit, through the screening facility and back to the existing conduit for discharge into the Reserved Channel. Referring to Figure B, appended, available sites include Marine Park or within the proposed Massport terminal facility. Construction of a permanent structure in the MDC-owned Marine Park would, most likely, raise public opposition as well as opposition from the MDC Parks and Recreation Division for similar reasons that were presented earlier in this chapter under the discussion of the Columbus Park site for the storage/containment facility. Discussions with Massport officials indicated that a site may be available where a facility could be located without significantly interfering with their proposed operation. However, if consent was obtained from Massport, the facility would cost an estimated $1.6 million, including engineering and contingencies (ENR 2900).

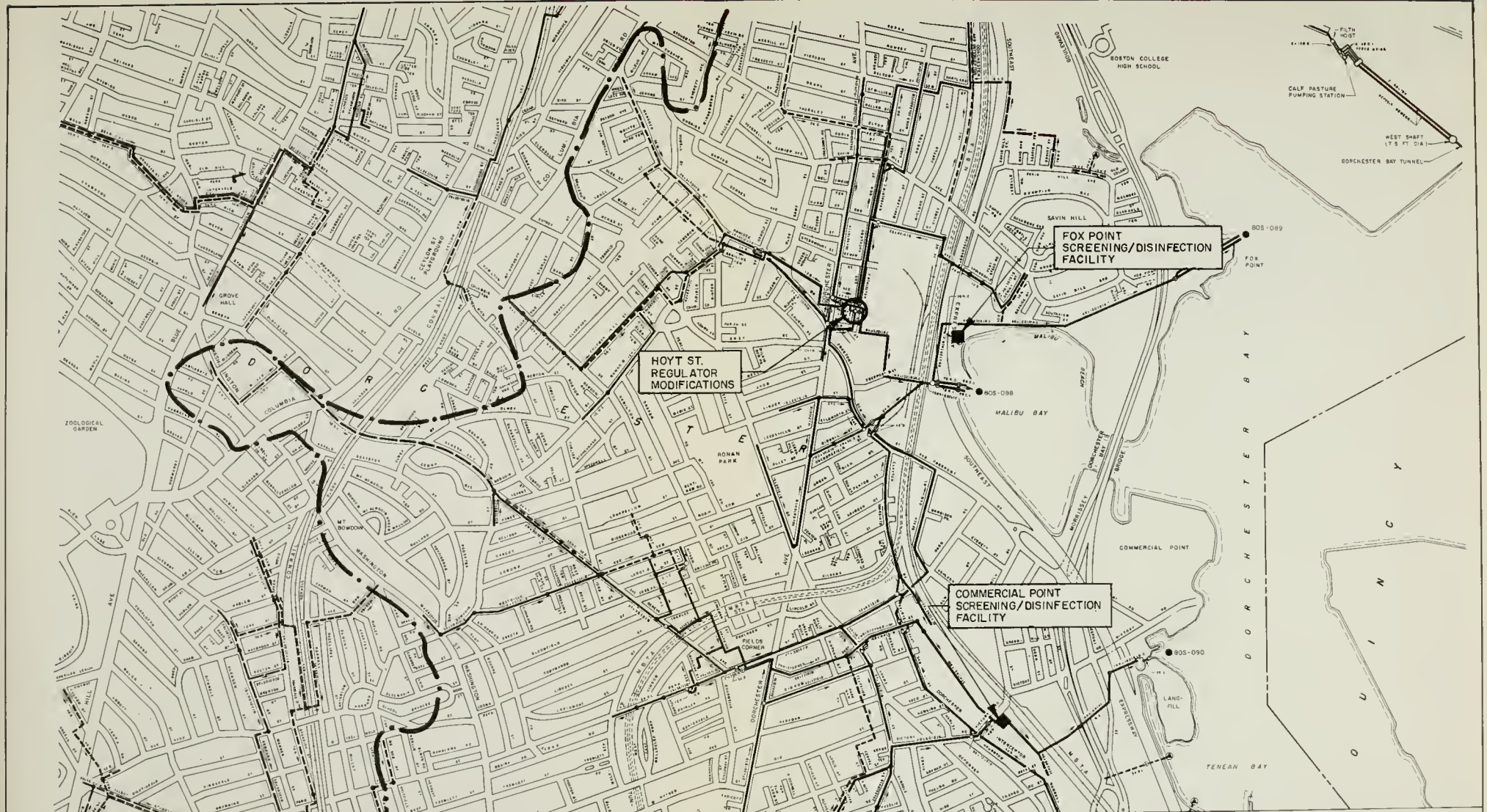
Based on the above discussion, treatment of CSO's generated from the area tributary to outlet BOS-080 is not considered to be warranted at this time when considering the nature and size of the tributary area, the water quality and use of the receiving water, and the estimated cost required to provide a minimum treatment level. However, it is recommended that the need for control of outlet BOS-080 CSO's be re-evaluated after the Inner Harbor Study Area's recommended Fort Point Channel CSO Treatment Facility has been in operation for at least a one-year period. At that time, the water quality improvement in the Reserved Channel by control of CSO outlets BOS-076, 078 and 079 can be assessed and the need for control of outlet BOS-080 CSO's determined.

Dorchester

The Dorchester portion of the study area includes the areas tributary to CSO outlets BOS-088 (Fox Point) and BOS-090 (Commercial Point), and the 118 acre separated sewer area tributary to the Pine Neck Creek storm drain (Figure XI-1). The major structural recommendations include modifications to the existing Hoyt Street regulator, screening/disinfection facilities on the two CSO conduits leading to the above outlets, and relocation of the Pine Neck Creek storm drain's outlet. Figure XII-20 and XII-21 show the locations of these facilities. The following sections present preliminary design criteria and a description, including preliminary plans, of each recommendation.

Hoyt Street Regulator Modifications. The purpose and resulting benefit of this recommendation was discussed in Chapter XI. Referring to Figure XI-16 of that chapter, it was suggested that a manually operated control gate be installed in the adjacent tide gate manhole for preventing the occurrence of overflows from this regulator, while also allowing for relief of the Dorchester interceptor during extreme flow conditions. It is recommended that a sluice gate, equipped with a manual operator, located at ground level be utilized for this purpose. A sluice gate is preferred over a slide gate since it provides a more positive closure and, due to its rugged construction, is more applicable for service in a potentially corrosive environment. In addition, due to the expected infrequent use of the gate, there is potential for restriction in the operation of a slide gate from the buildup of solids within the guide grooves.

It is recommended that the existing Hoyt Street regulator be modified by constructing a new sluice gate chamber and access manhole to replace the existing upstream tide gate section of the diversion conduit, as shown on Figure XII-22. As indicated on the plan and profile of the existing structure, the top section of the upstream tide gate manhole and side walls of the present box conduit would be removed to the top elevation of the base slab. New reinforced concrete side walls would be cast upon the existing base slab, and would extend to ground elevation resulting in a new chamber for the proposed sluice gate. Concrete fill would be placed in the existing conduit's invert, providing a sloping shelf rising 6-inches above the existing side-weir on the Dorchester Interceptor. The proposed gate sill elevation would be 103.5 ft. (MDC base).

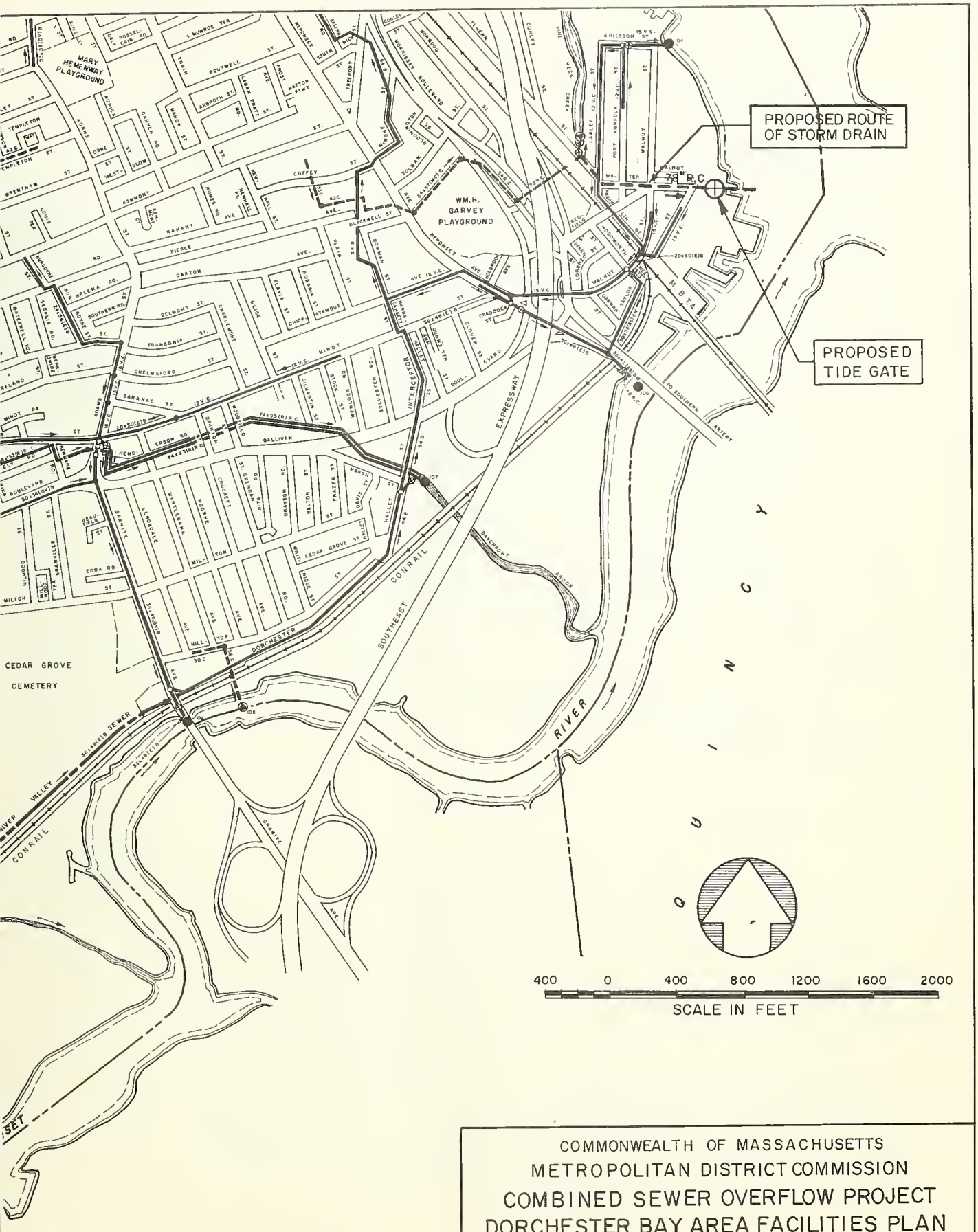



COMMONWEALTH OF MASSACHUSETTS

METROPOLITAN DISTRICT COMMISSION

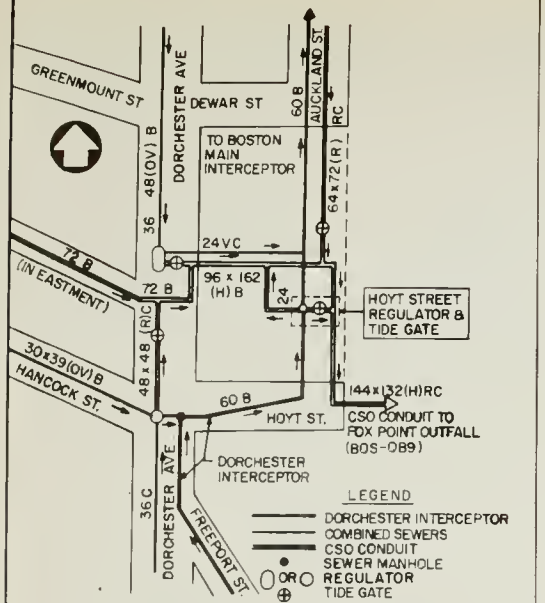
COMBINED SEWER OVERFLOW PROJECT
DORCHESTER BAY AREA FACILITIES PLAN

RECOMMENDED STRUCTURAL FACILITIES FOR DORCHESTER
FIG. XII-20



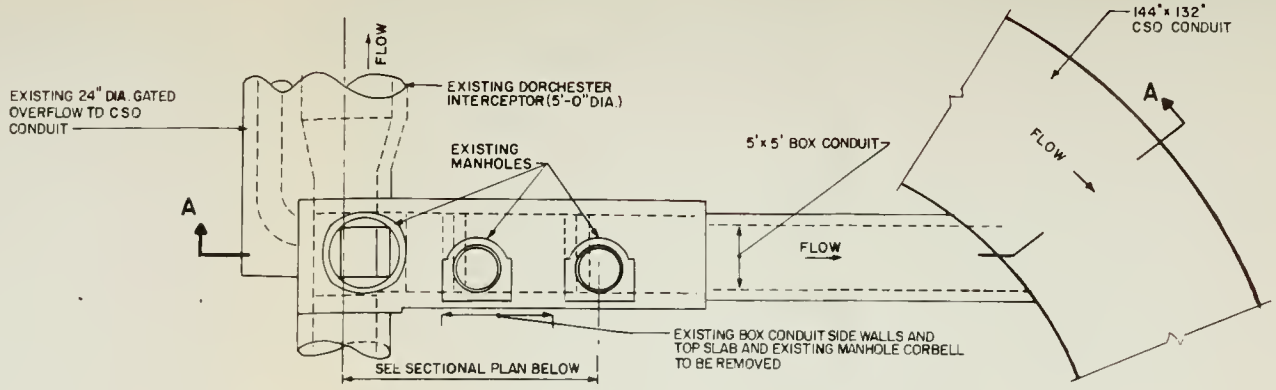

COMMONWEALTH OF MASSACHUSETTS
METROPOLITAN DISTRICT COMMISSION
COMBINED SEWER OVERFLOW PROJECT
DORCHESTER BAY AREA FACILITIES PLAN

RECOMMENDED PINE NECK CREEK
STORM DRAIN RELOCATION
FIG. XII-21

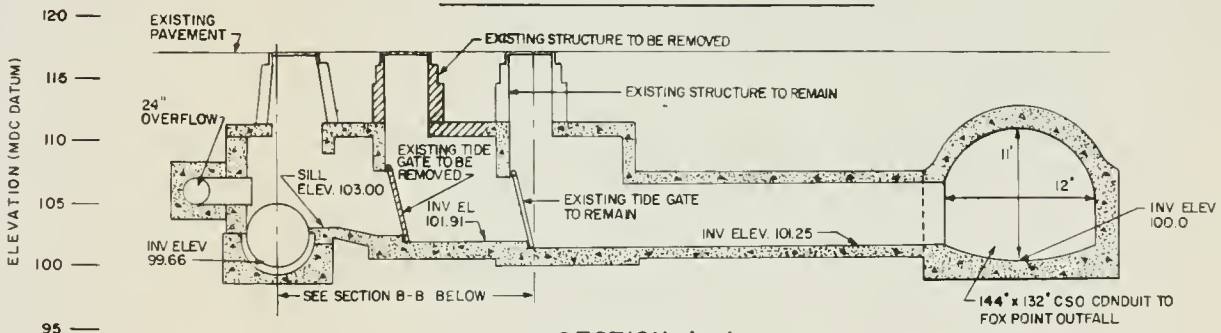



SCHEMATIC PLAN OF SEWERAGE SYSTEM NEAR THE HOYT STREET REGULATOR

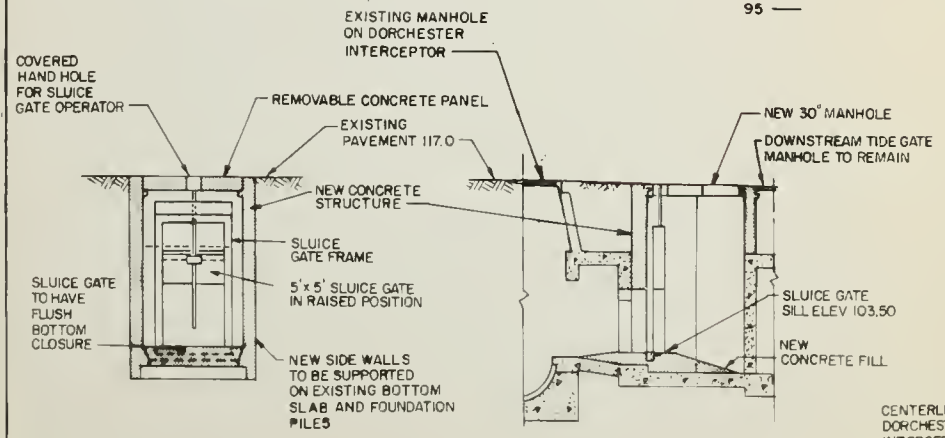
NOT TO SCALE


FLOW
EXISTING 24" DIA. GATED OVERFLOW TO CSO CONDUIT
EXISTING DORCHESTER INTERCEPTOR (5'-0" DIA.)
EXISTING MANHOLES
5'x5' BOX CONDUIT
144"x132" CSO CONDUIT
A
FLOW
FLOW
EXISTING BOX CONDUIT SIDE WALLS AND TOP SLAB AND EXISTING MANHOLE CORBELL TO BE REMOVED
SEE SECTIONAL PLAN BELOW

PLAN OF EXISTING CSO REGULATOR



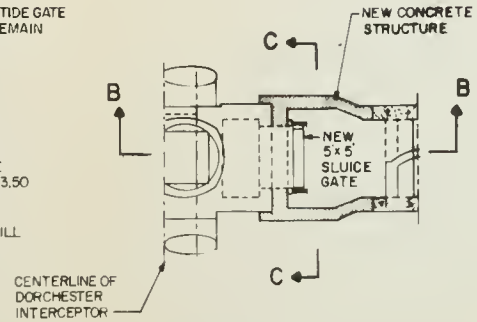

SECTION A-A


EXISTING MANHOLE ON DORCHESTER INTERCEPTOR
COVERED HAND HOLE FOR SLUICE GATE OPERATOR
REMOVABLE CONCRETE PANEL
EXISTING PAVEMENT 117.0
NEW CONCRETE STRUCTURE
SLUICE GATE FRAME
5'x5' SLUICE GATE IN RAISED POSITION
SLUICE GATE TO HAVE FLUSH BOTTOM CLOSURE
NEW SIDE WALLS TO BE SUPPORTED ON EXISTING BOTTOM SLAB AND FOUNDATION PILES
NEW 30" MANHOLE
DOWNSTREAM TIDE GATE MANHOLE TO REMAIN
SLUICE GATE SILL ELEV 103.50
NEW CONCRETE FILL

SECTION C-C

SECTION B-B

PROPOSED ALTERATIONS




SECTIONAL PLAN OF PROPOSED ALTERATIONS

LEGEND

NEW WORK

EXISTING STRUCTURE (TO BE REMOVED)

10 0 10 20

SCALE IN FEET

COMMONWEALTH OF MASSACHUSETTS

METROPOLITAN DISTRICT COMMISSION

COMBINED SEWER OVERFLOW PROJECT

DORCHESTER BAY AREA FACILITIES PLAN

HOYT STREET REGULATOR MODIFICATIONS

FIG. XII-22

The new gate wuld be a 60-in. x 60-in. self-contained, flush bottom closure sluice gate equipped with side wedges and a non-rising stem. The mounting flange would be bolted to the new concrete wall as shown on Figure XII-22. Downstream of the new gate, concrete fill would be installed on the base of the manhole to provide a sloping invert leading to the existing downstream tide gate chamber.

The roof slab would be constructed at existing grade (117.0 MDC base). The portion of this slab located above the sluice gate would be removable (supported by the side walls of the new chamber) to permit access for heavy maintenance and/or future replacement. A 30-in. diameter manhole would also be provided in the roof slab to provide access for routine inspection and maintenance. Within the removable slab, a small steel hatch would be located directly above the sluice gate stem operating nut. The gate would be operated using either a portable, gasoline-powered operator or a standard gate valve "key". The portable operator is recommended to facilitate operation especially during adverse weather conditions. It would also allow a more rapid opening and closing operation.

The total estimated capital cost of the regulator modifications at June, 1979 price levels (ENR 2900) is summarized as follows:

| | |
|---|---|
| Excavation and Sheeting | $20,000 |
| Concrete | 25,000 |
| Equipment | 25,000 |
| Subtotal | 70,000 |
| Engineering and Contingencies (30%) | 20,000 |
| Total | $90,000 |

Fox Point and Commercial Point Screening/Disinfection Facilities. The two Dorchester screening/disinfection facilities have identical design criteria and essentially the same layout, thus a discussion of both will be presented in this section.

Design Storm Selection. The design storm selected for each facility was the 1-year, 6-hour synthetic storm as developed by the Charles River Basin consultant. This control level will provide a minimum of coarse and fine screening of all CSO's generated from the Dorchester portion of the study area that do not exceed design storm flow rates. Overflows occurring during the summer bathing season would also be chlorinated. This lower frequency design storm was selected in lieu of the South Boston 3-month, 4-hour control level, primarily because the existing CSO outfall conduits, which have a 15-year design storm capacity will be utilized as part of the screening facilities (i.e., inlet and outlet). The conduit size required to convey flow from the existing CSO outfall, through the screening/disinfection facility and back to the outfall is the only major component affected by the difference in peak flow rate

from the two synthetic storms. The difference in conduit size would also affect the size of the channel within the facility and, accordingly, the size of the required control gates and the width of the screens, both of which are located within the channel. The size differential between the storms is minimum, since installation and not materials is the major cost associated with the new conduit. The overall cost difference between a 1-year, 6-hour and 3-month, 4-hour facilities is less than 10 percent. Considering the cost-effectiveness of screening/disinfection as compared to other alternatives (Chapter XI), an additional 10 percent expenditure to achieve a slightly higher control level is warranted.

Hydraulic Design Criteria. The computer model SWMM was utilized to generate 1-year, 6-hour storm hydrographs for the Fox Point and Commercial Point CSO conduits at the locations of the proposed screening/disinfection facilities. Since the downstream portion of the Commercial Point conduit was discovered in the field program to have two to three feet of sediment, runs were made with and without the sediment buildup (i.e., a "clean" conduit). Under all conditions simulated, it was assumed that the existing earthern dike at the BOS-090 outlet (see Chapter IV), that currently imposes a hydraulic restriction to flow emanating from the CSO conduit, would be removed. Removal of the dike will be discussed in greater detail later in this chapter. No significant, hydraulically restricting deposition was observed in the Fox Point CSO conduit during field investigation, thus SWMM was run assuming a "clean" conduit.

Two tidal conditions were also simulated with SWMM in order to determine the differences in conduit flow, velocity and depth. Runs were made with the 1-year, 6-hour storm beginning at high tide, and also with the storm beginning such that the peak of the hyetograph coincides with high tide. The results of the SWMM modeling runs for each condition are summarized on Table XII-4. As shown, the maximum flowrates for each conduit occurred when the storm began at high tide (274.6 cfs at 3:10 for Commercial Point and 161.1 cfs at 3:20 for Fox Point). For this condition, the peak of the storm occurred approximately at the mid point of ebbing tide, thus as the storm intensified and the conduit flow increased, the downstream hydraulic restraint imposed by the tide gradually lessened allowing a higher flowrate to the bay. Lower flowrates and greater depths of flow were evident when the storm peak coincided with high tide. This reflects the downstream hydraulic restraint imposed by the tidal condition. The restraint requires a "buildup" of flow in the conduit upstream of the tide gate to a depth greater than the tide elevation before the tide gate will open to allow discharge to the bay. As shown, the maximum flow depths under this condition were 9.0 ft. and 9.4 ft. for the Commercial Point and Fox Point conduits, respectively. Comparison of runs made with and without the sediment bed in the Commercial Point conduit resulted in no significant differences in flow rate or depth of flow. However, these runs were made with existing sediment depths which ranged between two to three feet. As discussed subsequently in this chapter, it is recommended that sediment be removed to prevent further accumulation which could impose a more significant restriction to flow. Once the conduit is clean and the existing earthen dike at the BOS-090 outlet is removed, the conduit's current flow velocities will increase and the potential for sediment deposition would be significantly reduced.

TABLE XII-4 RESULTS OF SWMM FOR 1-YEAR, 6-HOUR STORM
FOR FOX POINT AND COMMERCIAL POINT CSO CONDUITS

| Time After Start of Storm | COMMERCIAL POINT CSO CONDUIT(1) | | | | | | FOX POINT CSO CONDUIT(2) | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | Storm Start At High Tide(3) | | | Storm Peak at High Tide | | | Storm Start at High Tide | | | Storm Peak at High Tide | | |
| | Flow (cfs) | Vel. (fps) | Depth (ft) | Flow (cfs) | Vel. (fps) | Depth (ft) | Flow (cfs) | Vel. (fps) | (Depth) (ft) | Flow (cfs) | Vel. (fps) | Depth (ft) |
| 1:00 | 0.0 | 0.0 | 0.0 | 0.0 | 0.0 | 0.0 | 0.0 | 0.0 | 0.2 | 0.0 | 0.0 | 0.2 |
| 1:30 | 0.0 | 0.0 | 0.0 | 0.1 | 0.1 | 0.6 | 0.8 | 0.1 | 0.8 | 0.8 | 0.1 | 0.8 |
| 2:00 | 2.4 | 0.3 | 0.9 | 14.0 | 0.6 | 1.8 | 2.8 | 0.2 | 1.7 | 2.8 | 0.2 | 1.7 |
| 2:30 | 13.8 | 0.5 | 1.8 | 23.9 | 0.6 | 2.9 | 3.7 | 0.2 | 2.3 | 3.7 | 0.2 | 2.3 |
| 3:00 | 52.9 | 1.1 | 3.8 | 64.6 | 1.1 | 4.7 | 16.3 | 0.5 | 4.3 | 16.3 | 0.5 | 4.3 |
| 3:10 | 274.6 | 4.5 | 5.5 | 208.7 | 2.3 | 7.4 | 81.9 | 1.5 | 6.6 | 47.5 | 0.9 | 6.8 |
| 3:20 | 249.7 | 4.5 | 5.0 | 162.7 | 1.6 | 8.3 | 161.1 | 3.4 | 6.3 | 41.8 | 0.6 | 8.0 |
| 3:30 | 184.1 | 3.7 | 4.4 | 117.3 | 1.1 | 8.6 | 136.6 | 3.1 | 5.8 | 43.6 | 0.6 | 8.7 |
| 4:00 | 96.8 | 2.6 | 3.1 | 66.7 | 0.6 | 9.0 | 77.9 | 2.3 | 4.5 | 27.3 | 0.4 | 9.2 |
| 4:30 | 54.9 | 2.2 | 2.1 | 38.5 | 0.3 | 9.0 | 55.0 | 2.1 | 3.5 | 21.9 | 0.3 | 9.4 |
| 5:00 | 35.2 | 2.2 | 1.6 | 27.6 | 0.2 | 9.0 | 42.3 | 2.1 | 2.8 | 4.6 | 0.1 | 6.8 |
| 5:30 | 27.0 | 2.5 | 1.4 | 22.2 | 0.2 | 9.0 | 33.3 | 2.1 | 2.3 | 5.7 | 0.1 | 7.0 |
| 6:00 | 22.6 | 2.5 | 1.3 | 20.1 | 0.2 | 9.0 | 26.8 | 1.9 | 2.0 | 41.8 | 0.8 | 6.6 |
| 6:30 | 19.9 | 2.4 | 1.2 | 38.7 | 0.4 | 8.1 | 18.5 | 1.4 | 1.8 | 40.8 | 0.9 | 5.5 |
| 7:00 | 10.8 | 1.2 | 1.0 | 37.0 | 0.4 | 7.1 | 4.3 | 0.3 | 1.7 | 23.0 | 0.6 | 4.5 |
| 7:30 | 3.5 | 0.3 | 0.8 | 22.2 | 0.3 | 6.2 | 1.7 | 0.1 | 1.7 | 18.0 | 0.6 | 3.6 |
| 8:00 | 1.7 | 0.1 | 0.9 | 18.3 | 0.3 | 5.5 | 1.2 | 0.1 | 1.7 | 14.9 | 0.6 | 2.9 |

Notes: (1) 14'-0" wide x 11'-6" high horseshoe-shaped reinforced concrete conduit.
(2) 8'-0" wide x 10'-0" high rectangular reinforced concrete conduit.
(3) Includes existing sediment bed in conduit, assumed to be worst case condition.

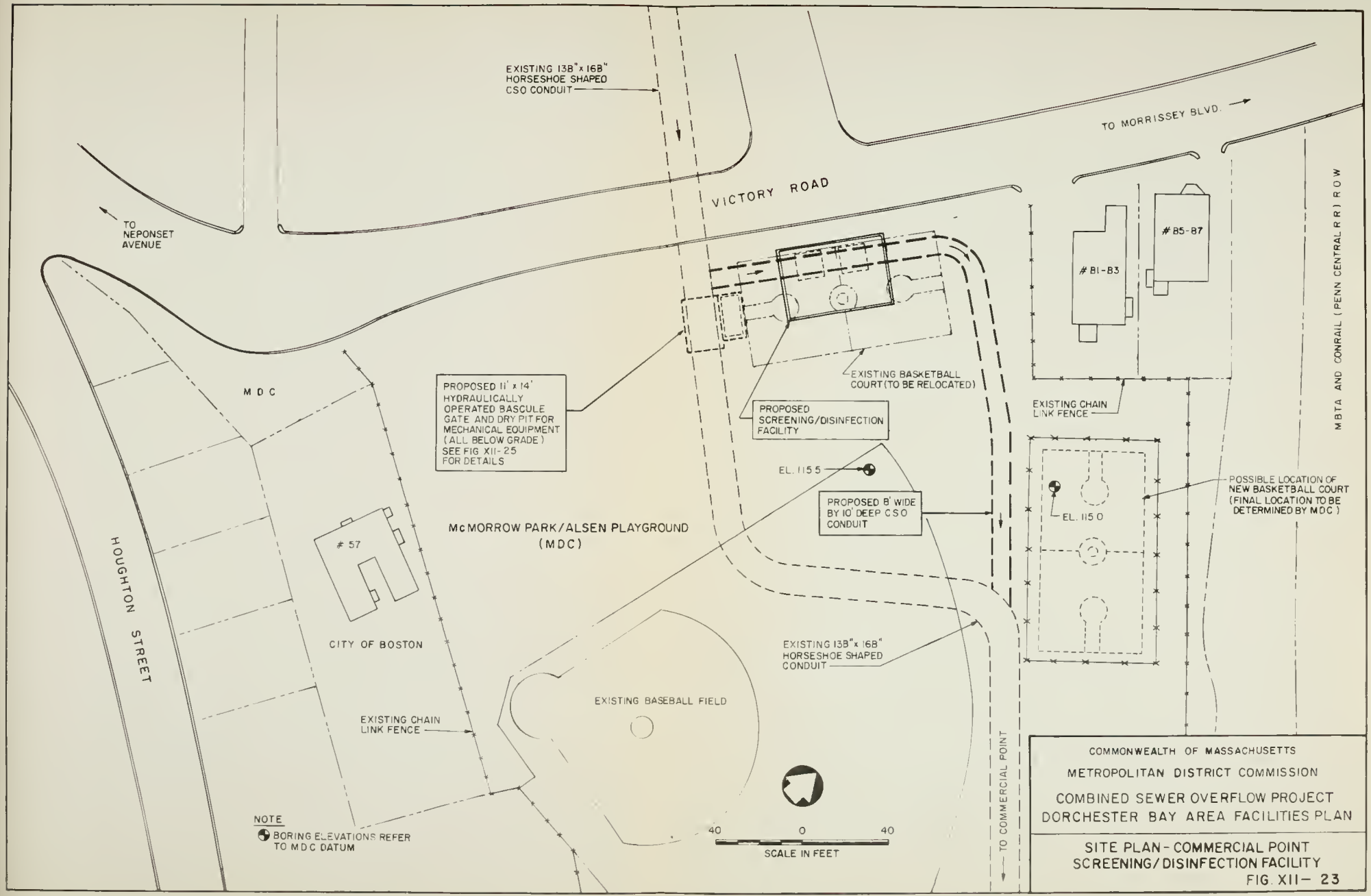
The modeling results for the 1-year, 6-hour storm indicate maximum flowrates of about 275 cfs for the Commercial Point facility, and about 160 cfs for the Fox Point facility. For preliminary design, flowrates of 300 cfs and 180 cfs for the Commercial Point and Fox Point facilities (i.e., 10 percent increase) respectively, were adopted to allow for modeling inaccuracies. For the Commercial Point facility, an 8' wide by 10' high rectangular box conduit would be required to convey flow from the existing 14' wide x 11' high horseshoe-shaped CSO conduit to the screening/disinfection facility and back to the existing conduit. The slope of the new conduit would match the slope of the existing CSO conduit which is 0.00067 ft/ft. An 8' wide by 10' high rectangular box conduit is also required for the Fox Point facility. This conduit would be identical in size and configuration to the exiting Fox Point CSO conduit which has zero slope (i.e., laid flat) and relies on the upstream system's hydraulic gradient for flow conveyance. Hydraulic analysis indicates that a similar size conduit is required to pass 1-year, 6-hour storm flows through the screening/disinfection facility during a high tide coincident with storm's peak. This is evident in Table XII-4, where the maximum depth of flow in the existing CSO conduit is 9.4 ft under this tidal condition.

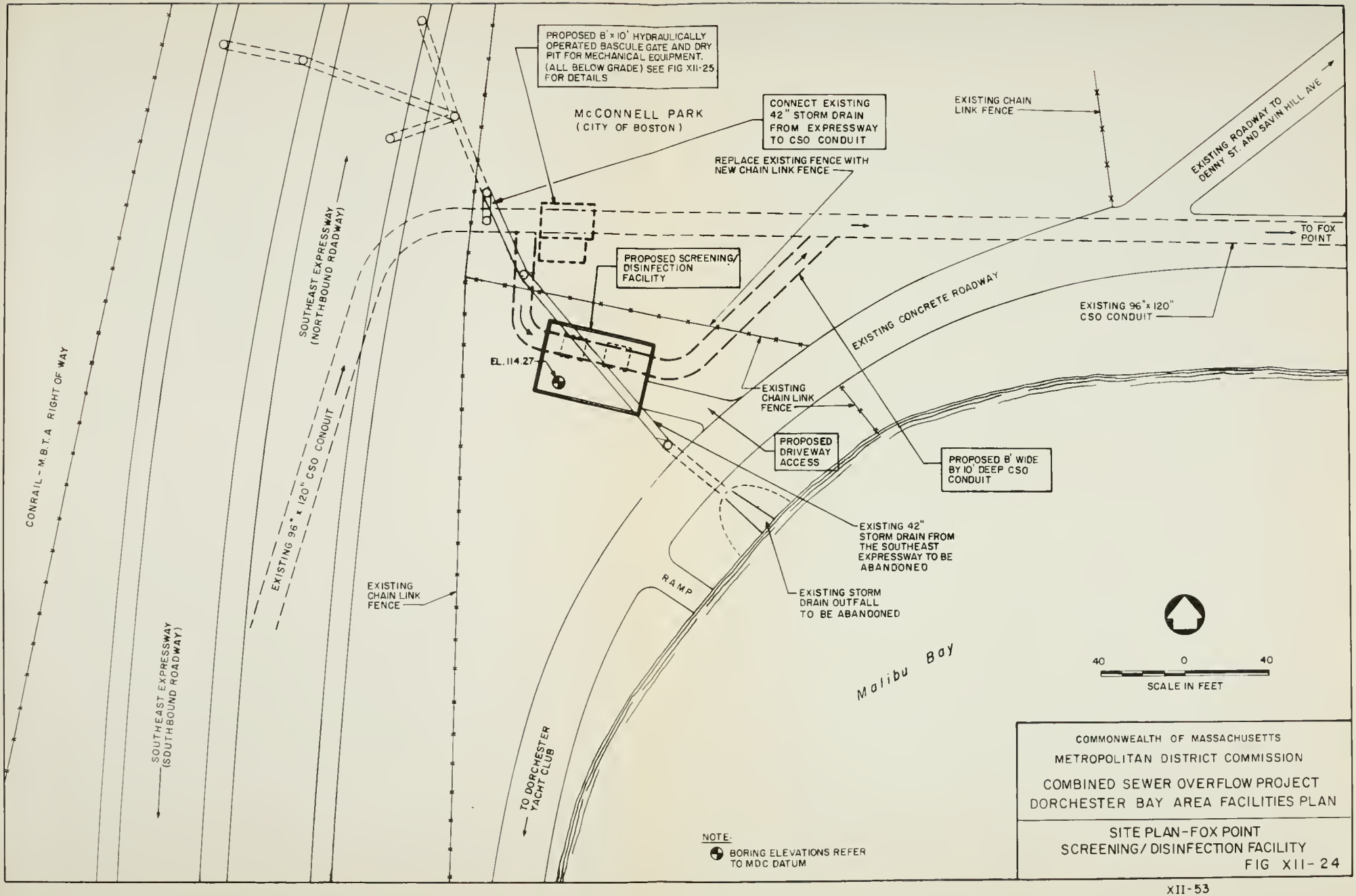
Summary of Design Criteria. A summary of the design criteria for the Commercial Point and Fox Point screening/disinfection facilities that was used for preliminary design is presented in Table XII-5. Discussed in subsequent sections are the major components of the facilities.

Siting and General Layout. Site plans for the Commercial Point and Fox Point screening/disinfection facilities are shown on Figures XII-23 and XII-24, respectively. The Commerical Point facility would be located along Victory Road in the MDC-owned McMorrow Park at the site of an existing basketball court. An alternate location between the basketball court and a baseball field was also considered, however, the selected site, which was suggested by the MDC Parks and Recreation Division, would result in the least long-term disruption to normal park activities since it would be more readily accessible from Victory Road. A new basketball court would be constructed at an alternate location to be determined by the MDC, and would be completed prior to demolition of the existing court to assure minimal disruption to normal park activities. As shown on Figure XII-23, a hydraulically powered bascule (tipping) gate would be constructed within the existing CSO conduit for diverting flow to the facility, and for by-passing flow in excess of design capacity. Flow would be conveyed to the facility through a new 8' by 10' rectangular box culvert, and pass through the facility where the waste stream would be screened. Chlorination would be employed during the summer months only. The treated waste stream would be conveyed back to the existing CSO conduit through the downstream section of the new conduit.

TABLE XII-5 SUMMARY OF DESIGN CRITERIA FOR COMMERCIAL POINT AND FOX POINT SCREENING/ DISINFECTION FACILITIES

| CRITERIA | COMMERCIAL POINT FACILITY | FOX POINT FACILITY |
|---|---|---|
| Tributary Area, acres | | |
| . Combined areas | 541 | 326 |
| . Separate areas | 340 | 59 |
| . Total area | 881 | 385 |
| Design Storm | 1-yr, 6-hr | 1-yr, 6-hr |
| Design Flow, cfs (mgd) | 300 (194) | 180 (116) |
| Existing CSO Conduit Size/Shape | 14'-0"x11'-6" horseshoe | 8'-0"x10'-0" rectangle |
| Existing CSO Conduit Slope (ft/ft) | 0.00067 | 0.0 |
| New CSO Diversion Conduit Size (rectangular shape) | 8'-0"x10'-0" | 8'-0"x10'-0" |
| New CSO Diversion Conduit Slope (ft/ft) | 0.00067 | 0.0 |
| Coarse Screen Bar Spacing, in. | 1.50 | 1.50 |
| Fine Screen Bar Spacing, in. | 0.75 | 0.75 |
| Type of Screen Adopted for Preliminary Design | catenary | catenary |
| Estimated Headloss through both Screens (clean) at Design flow, in. | 4-7 | 4-7 |
| Estimated Volume of Screenings from Design Storm, cu. ft. | | |
| . Coarse | 15 | 10 |
| . Fine | 30 | 22 |
| . Total | 45 | 32 |
| Estimated Total Annual Volume of Screenings, cu. ft. | 1300 | 380 |
| Estimated Total Annual Waste Stream Volume through Facility, mil. gal. | 144.0 | 40.6 |
| Chlorine Dosage, mg/l | 15 | 15 |
| Chlorine Detention Time, min. | | |
| . Peak flow | 10 | 17 |
| . Average flow | 17 | 33 |
| Estimated Annual Volume of Sodium Hypochlorite required (12% Solution), gal. | 5,000 | 1,500 |
| Building Size | 50'x34'x18' high | 50'x34'x16' high |


EXISTING 138"x 168" HORSESHOE SHAPED CSO CONDUIT
TO MORRISSEY BLVD.
VICTORY ROAD
TO NEPONSET AVENUE
MBTA AND CONRAIL (PENN CENTRAL R R) R O W
#81-83
#85-87
EXISTING BASKETBALL COURT (TO BE RELOCATED)
PROPOSED 11' x 14' HYDRAULICALLY OPERATED BASCULE GATE AND DRY PIT FOR MECHANICAL EQUIPMENT (ALL BELOW GRADE) SEE FIG XII-25 FOR DETAILS
PROPOSED SCREENING/DISINFECTION FACILITY
EXISTING CHAIN LINK FENCE
EL. 115.5
PROPOSED 8' WIDE BY 10' DEEP CSO CONDUIT
EL. 115.0
POSSIBLE LOCATION OF NEW BASKETBALL COURT (FINAL LOCATION TO BE DETERMINED BY MDC)
M D C
HOUGHTON STREET
#57
CITY OF BOSTON
McMORROW PARK/ALSEN PLAYGROUND (MDC)
EXISTING 138"x 168" HORSESHOE SHAPED CONDUIT
EXISTING BASEBALL FIELD
EXISTING CHAIN LINK FENCE
TO COMMERCIAL POINT
NOTE
BORING ELEVATIONS REFER TO MDC DATUM
40 0 40
SCALE IN FEET
COMMONWEALTH OF MASSACHUSETTS
METROPOLITAN DISTRICT COMMISSION
COMBINED SEWER OVERFLOW PROJECT
DORCHESTER BAY AREA FACILITIES PLAN
SITE PLAN - COMMERCIAL POINT SCREENING/DISINFECTION FACILITY
FIG. XII- 23


PROPOSED 8' x 10' HYDRAULICALLY OPERATED BASCULE GATE AND DRY PIT FOR MECHANICAL EQUIPMENT. (ALL BELOW GRADE) SEE FIG XII-25 FOR DETAILS
McCONNELL PARK (CITY OF BOSTON)
CONNECT EXISTING 42" STORM DRAIN FROM EXPRESSWAY TO CSO CONDUIT
EXISTING CHAIN LINK FENCE
REPLACE EXISTING FENCE WITH NEW CHAIN LINK FENCE
EXISTING ROADWAY TO DENNY ST. AND SAVIN HILL AVE
TO FOX POINT
PROPOSED SCREENING/ DISINFECTION FACILITY
EXISTING CONCRETE ROADWAY
EXISTING 96"x 120" CSO CONDUIT
EL. 114.27
EXISTING CHAIN LINK FENCE
PROPOSED DRIVEWAY ACCESS
PROPOSED 8' WIDE BY 10' DEEP CSO CONDUIT
EXISTING 42" STORM DRAIN FROM THE SOUTHEAST EXPRESSWAY TO BE ABANDONED
EXISTING STORM DRAIN OUTFALL TO BE ABANDONED
RAMP
EXISTING CHAIN LINK FENCE
CONRAIL - M.B.T.A RIGHT OF WAY
SOUTHEAST EXPRESSWAY (NORTHBOUND ROADWAY)
EXISTING 96" x 120" CSO CONDUIT
SOUTHEAST EXPRESSWAY (SOUTHBOUND ROADWAY)
TO DORCHESTER YACHT CLUB
Malibu Bay
40 0 40
SCALE IN FEET
NOTE:
BORING ELEVATIONS REFER TO MDC DATUM
COMMONWEALTH OF MASSACHUSETTS
METROPOLITAN DISTRICT COMMISSION
COMBINED SEWER OVERFLOW PROJECT
DORCHESTER BAY AREA FACILITIES PLAN
SITE PLAN-FOX POINT
SCREENING/ DISINFECTION FACILITY
FIG XII-24

The screening and disinfection equipment and apurtenances would be housed in a concrete block and brick building with a flat roof of prestressed concrete double-tee beam construction. The dimensions of the structure would be approximately 50-ft by 34-ft in plan area, and about 18-ft high. A portion of the existing paved roadway into the playground from Victory Road and a new paved driveway from this existing roadway to the west side of the new structure, would be used for vehicle access to the facility, primarily for screenings removal. With the exception of the superstructure, all new construction would be located at or below existing ground surface elevation.

The Fox Point screening/disinfection facility would be located in the currently vacant lot bordered by McConnell Park (City of Boston) on the north, the Southeast Expressway on the west, Malibu Bay on the east and the Dorchester Yacht Club on the south. The lot is owned by the Massachusetts Department of Public Works and is leased to the Dorchester Yacht Club which sometimes utilized it for boat storage. Discussions with these concerns indicated that there should be no significant problems in obtaining this land and utilizing it as a site for the superstructure. As shown on Figure XII-24, the facilities required for diverting flow from the existing CSO conduit to the screening/disinfection facility and back to the conduit would be identical to those described for the Commercial Point facility. The superstructure size and configuration would also be the same except the building height which is 16-ft. As shown, the existing 42-in. diameter storm drain serving the Southeast Expressway would be tied into the new facility.

Access to the facility would be from Springdale Street extended, which leads to the paved roadway located along Malibu Beach. This roadway also serves as access to Dorchester Yacht Club. A paved driveway from this roadway to the east side of the superstructure would provide vehicle access, primarily for screenings removal. It should be noted that a portion of the new diversion conduit construction would occur at the southern end of McConnell Park. Officials from the City of Boston Parks and Recreation Department suggested that the major portion of this construction be completed during the park's minimal use period (i.e., late fall to early spring) to minimize disruption to normal park activities. No above ground structures would be located in the park upon completion of construction.

Flow Diversion and Regulation. As discussed earlier in this section, flow diversion is required to convey the waste stream from the existing CSO conduit to the screening/disinfection facilities. The capability of bypassing flows that exceed the design capacity of the facility must also be provided. To accomplish both flow diversion and regulation, a bascule (i.e., tipping) gate installed within the existing CSO conduit is recommended at each facility. Sluice gates were also considered, but an above ground structure would be required to house the gate when it is in the open position since, at each facility, there is insufficient cover over the existing CSO conduits to allow a completely below-grade operation. Besides allowing all controls to be housed in chambers below grade, the bascule gate acts as an adjustable overflow weir which

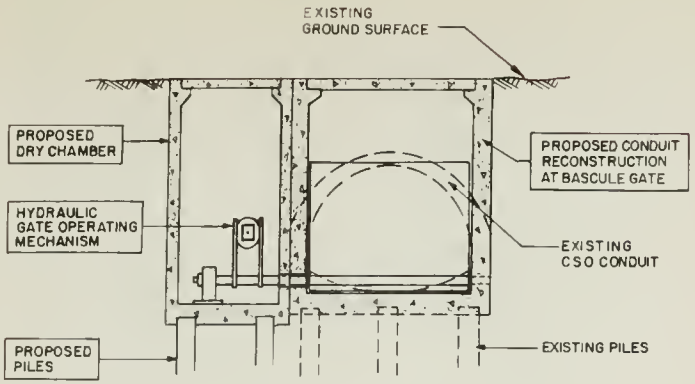
bypasses the upper portion of the waste stream while diverting the lower portion to the screening/disinfection facility. Considering the relatively low flow velocities in the existing CSO conduits, the lower portion of the waste stream would, most likely, contain a higher concentration of pollutants due to sedimentation.

As shown on Figures XII-23 and XII-24, at each facility the bascule gate would be constructed on the existing CSO conduit just downstream of the new 8' by 10' diversion conduit. The gate would normally be in the full vertical position, thus diverting all flow through the facility. When flow exceeds the design flow of the facility, the angular position of the gate (which would act as variable height overflow weir by tipping in the downstream direction) would be automatically adjusted, using the depth-of-flow sensing system located in the screening channel. The gate would be positioned by a hydraulic cylinder, and powered by a hydraulic accumulator system located in the main building. The gate would pivot on trunnions at the invert of the existing CSO conduit. In the event of a power interruption, the gates would automatically be driven by pressure retained in the hydraulic system accumulator tank to the wide-open (or minimum conduit depth) position. Details of this installation are shown on Figure XII-25.

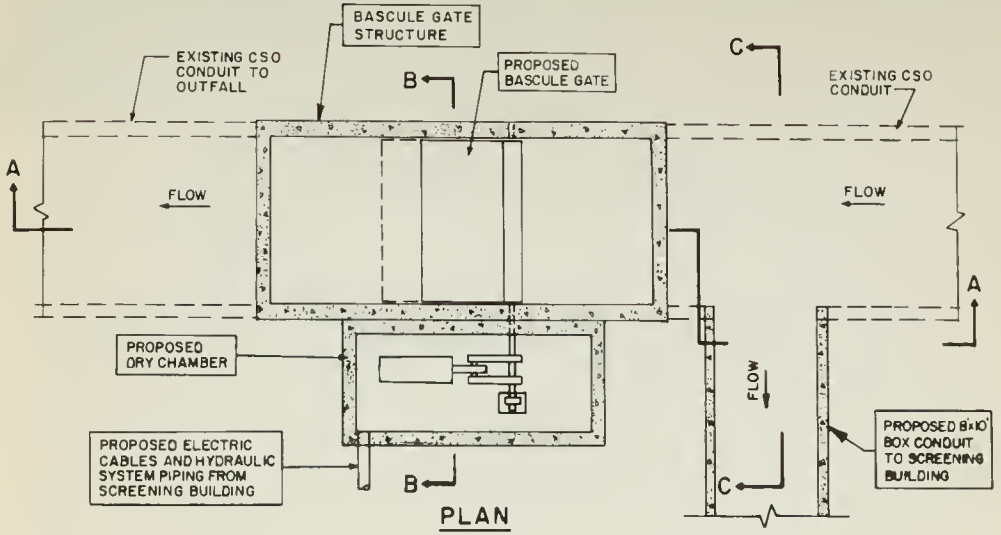
The structural work for the bascule gate installation at the Commercial Point facility would require partial demolition and reconstruction of about 25 linear feet of the existing 14' by 11'-6" horseshoe-shaped CSO conduit. During this construction, flow would be diverted through the proposed 8' x 10' diversion conduit which would have been completed. The new structural work would be built on the existing piling, and additional piles would be required to support the adjacent hydraulic cylinder chamber, all of which would be built at or below the existing ground elevation (Figure XII-25). Hydraulic lines, electric power for lights, heat and humidity control, and control wiring between the bascule gate chamber and the main building would be located in buried concrete-encased conduits.

Screening. As shown on Figure XII-26, the screening/disinfection facilities have been designed for the installation of "coarse" and "fine" screens in series. Considering the relatively high rates of flow and the extremely "flat" gradients within each system, especially during a high tide condition, selection of the type of screening mechanism was based, in part, on minimizing head loss through the units. Also considered in the selection was maintenance requirements, durability and dependability, operating history, and costs of the units.

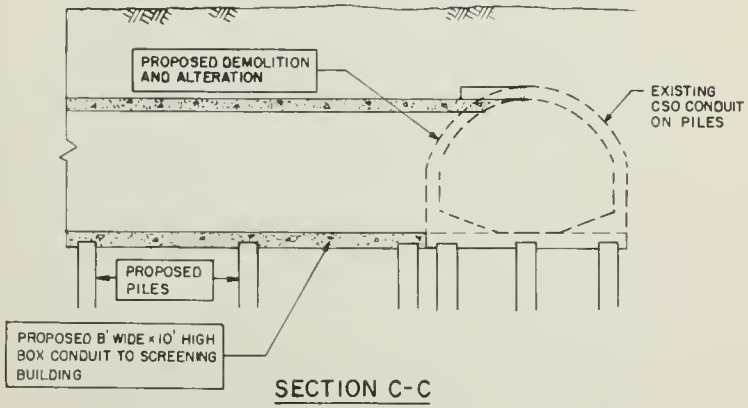
Two types of screening mechanisms were considered: (1) stationary bar screens, and (2) traveling woven wire media screens. The traveling woven wire media screens, which were considered for fine screening only, offer higher removal efficiencies due to the mesh design, however, operation and maintenance requirements are extremely high. Since the media is cleaned by applying a high velocity water jet spray, handling and disposal of this sidestream would greatly increase the complexity of the operation, as well as the required building size, operational requirments and, consequently, costs. In addition, the head loss through this unit



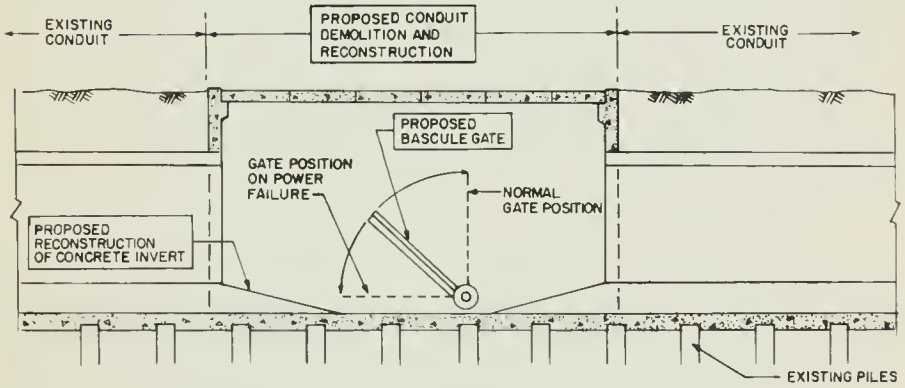

SECTION B-B




PLAN


PROPOSED DEMOLITION AND ALTERATION
EXISTING CSO CONDUIT ON PILES
PROPOSED PILES
PROPOSED 8' WIDE x 10' HIGH BOX CONDUIT TO SCREENING BUILDING

SECTION C-C




SECTION A-A

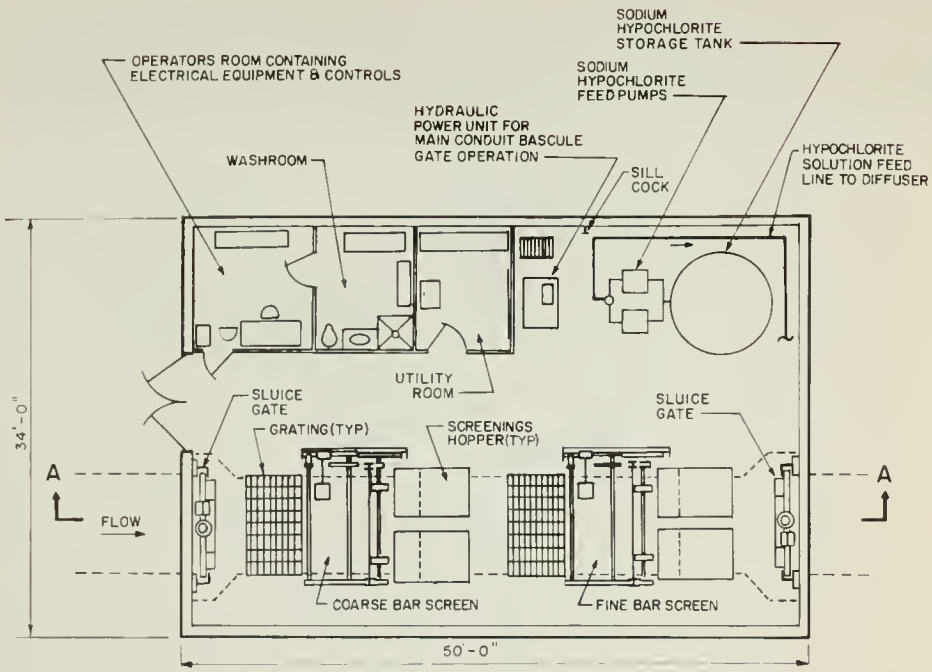
10 0 10

SCALE IN FEET

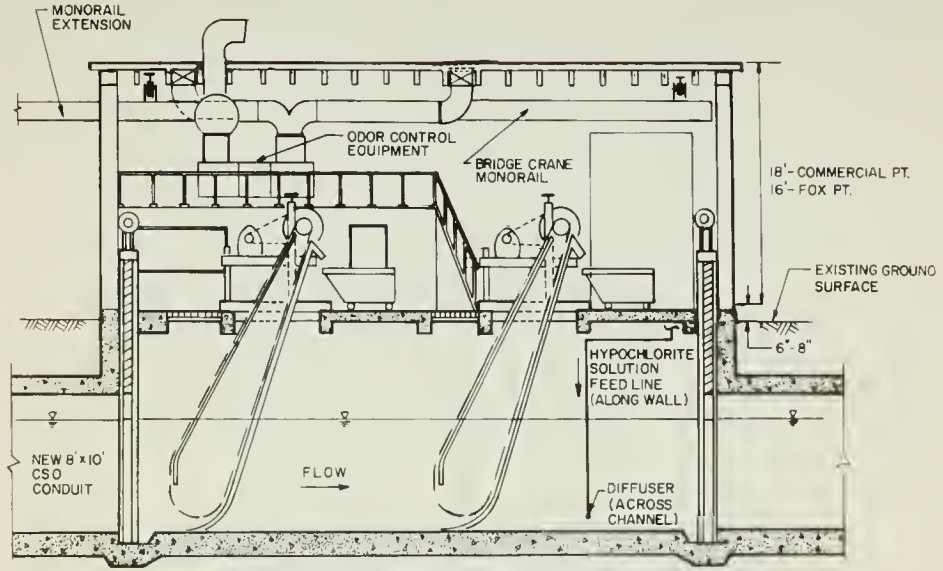
NOTE:
EXISTING CONDUITS,
COMMERCIAL POINT: 14' WIDE x 11.5 HIGH (HORSESHOE) - SEE FIG XII-23
FOX POINT: 8' WIDE x 10' HIGH (RECTANGULAR) - SEE FIG XII-24

COMMONWEALTH OF MASSACHUSETTS
METROPOLITAN DISTRICT COMMISSION
COMBINED SEWER OVERFLOW PROJECT
DORCHESTER BAY AREA FACILITIES PLAN

PROPOSED BASCULE GATE AT COMMERCIAL POINT AND FOX POINT SCREENING/ DISINFECTION FACILITIES (SCHEMATIC, PLAN AND SECTIONS) FIG. XII-25




PLAN




SECTION A-A

10 0 10 20

SCALE IN FEET

COMMONWEALTH OF MASSACHUSETTS

METROPOLITAN DISTRICT COMMISSION

COMBINED SEWER OVERFLOW PROJECT

DORCHESTER BAY AREA FACILITIES PLAN

COMMERCIAL POINT AND FOX POINT SCREENING/DISINFECTION FACILITIES

FIG. XII-26

is two to three times that of a stationary unit. Accordingly, this method of fine screening is not considered applicable for use at the two Dorchester facilities. The use of mechanically cleaned stationary bar screens would result in a less complex operation having relatively few moving parts. These units are less labor intensive, provide better flow conditions, and are extremely durable due to their rugged construction. The operation of the unit is not limited by velocities or depth of flow in the channel, thus they can handle flows generated by extremely low intensity storms as well as design storm flows. For these reasons, they are recommended for use at the two Dorchester screening/disinfection facilities for coarse and fine screening of CSO's. Three types of mechanically cleaned bar screens were considered:

1. Catenary type
2. Climber type
3. Bucket and cable type

Each type mechanically cleans the bar screen using a scraper mechanism that rides upwards along the bars, discharging the accumulated screenings into a hopper located at ground floor level. The catenary unit applies the catenary principle to the rake path, using the weight of heavy rakes and drive chains to form a catenary loop. Each side of the rake is connected to a chain and sprocket mechanism which is driven by a motor located on supports above the operating floor. The catenary loop cleans the entire face of the bar rack by traveling upwards along the bar screen and discharging the accumulated screenings into a storage hopper. Since the unit employs a looping concept of operation, it can adjust to surge accumulations without jamming or overloading.

A climber screen removes solids from the bar screen by means of a single, gear driven cleaning rake running in guide rails. The guide rails are located on either side of the channel and are part of the unit's self-supporting frame. As the mechanism decends the down-guide rail, the rake is held away from the bar screen. As the drive sprocket reaches the bottom and begins to climb the up-guide rail, the rake swings over and meshes with the bar screen. The rake then conveys the screenings up the face of the screen and discharges them into a storage hopper. Similar to the catenary screen, all components of the drive mechanism are located above the waste stream at the operating floor level.

One of the oldest methods used to mechanically clean bar screens employs a bucket and cable-type mechanism. The "bucket" consists of a metal plate, the length of which equals the width of the bar screens, with end plates and a rear plate (to prevent the loss of screenings), and teeth that are cut to match the bar openings. This "bucket" is raised up along the face of the screen by four cables driven by an overhead motor. The unit is designed to allow the bucket to discharge its contents into a storage hopper at the operating floor level. All moving parts are located above the waste stream at the operating floor level.

Each of the above units have successful operating histories. The climber screen is the most expensive of the units, and it requires the largest amount of headroom since the support frame containing the rake guides is completely above the water level. The catenary unit is currently the most widely used screening mechanism. It requires about one-half the head room of the climber screen since the drive chain rotates in a relatively compact area around the screen. Catenary units have been installed in several area pumping stations and wastewater treatment plants including several MDC and City of Boston facilities. All MDC headworks facilities (i.e., Columbus Park, Ward Street and Chelsea Creek) use catenary type units. Several of these installations report successful, long-term operating experience with extremely low maintenance requirements. The bucket and cable type screen is comparable in cost to the catenary unit, and has a similar successful operating history. Although there are a lower number of existing installations in this area as compared to catenary screens, they appear to be comparable to the catenary unit with regards to maintenance requirements, efficiency and long-term successful operation.

Accordingly, either unit would be applicable for coarse and fine screening at the two Dorchester screening/disinfection facilities. The climber screen is not recommended, since its high overhead space requirements would result in a superstructure that would be about 3-feet higher than required for either the catenary or bucket and cable screens. The cost of the climber screen is also slightly higher than either of these latter two screens.

Since catenary units are currently more widely used, and considering there are a larger number of successful and virtually maintenance-free installations within the area, they have been used for the preliminary design of the two Dorchester screening/disinfection facilities. The fact that the MDC has essentially standardized the use of catenary screens was also considered in the selection. It is recommended that these two screening units be reevaluated at the time of final design prior to final equipment selection, in order to incorporate into the decision-making process any operational difficulties or differences in costs that may occur in the interim period prior to final design.

As shown on Figure XII-26, the coarse and fine screens would be installed in series within the 8' by 10' CSO diversion conduit. The screens would be inclined approximately 30° from vertical, and consist of 5/8-in. x 5/16-in. x 1-3/4-in. trapezoidal shaped steel bars uniformly spaced to provide 1-1/2-in. clear openings for the coarse screen, and 3/4-in. clear openings for the fine screen. It is estimated that under peak flow conditions, the head loss through both screens would be in the range of 4 to 7 inches at each facility (assuming "clean" screens). The rakes would consist of steel plates with teeth cut to match the bar openings. They would be attached to the chain drive at approximately 7'-0" intervals. Operation of the scraper mechanism would be controlled by a timer and/or the head loss across the screen. The chains, consisting of welded steel, would operate at a speed of about 10-ft. per minute and be driven by an electric motor with a chain and sprocket connection to the drive shaft. This equipment would be located on supports at the operating floor level as shown on Figure XII-26.

The estimated volume of screenings generated by the 1-year, 6-hour synthetic design storm at each facility is summarized as follows:

| | Commercial Point Facility | Fox Point Facility |
|---|---|---|
| Coarse screenings, cu. ft. | 15 | 10 |
| Fine screenings, cu. ft. | 30 | 22 |
| Total screenings volume, cu. ft. | 45 | 32 |

The estimated total annual volume of screenings is 1300 cu. ft. from the Commercial Point facility, and 380 cu. ft. from the Fox Point facility. The screenings would be discharged from the rakes, at a height of about 5 to 6 feet above the operating floor into self-dumping hoppers.

It is recommended that a total of four standard size hoppers (54 cu. ft. capacity) be provided for each facility. This would allow for adequate storage of screenings in the event of back-to-back storm events. The hoppers would be handled by an overhead bridge crane, equipped with an electrically driven hoist and trolley. The crane would interlock with a monorail extending outside the building to facilitate the discharge of screenings into a dump truck. The hoppers would be emptied into the truck outside of the building to avoid operating the vehicle inside the building where potentially explosive gases might be present. All electrical equipment within the building would be of explosion-proof rating. The ultimate disposal of screenings will be discussed later in this chapter.

Disinfection. As discussed in Chapter XI, the waste stream passing through the two screening/disinfection facilities would be chlorinated during the summer bathing season. A 12% concentration of sodium hypochlorite solution would be utilized as the disinfecting agent, and would be delivered in bulk to each of the facilities. Based on SEMSTORM modeling results which assume the Hoyt Street regulator modifications are completed, the estimated annual volumes of combined sewer overflows (from combined areas) and separate stormwater runoff (from separated areas) expected to occur at each facility are summarized as follows:

| | Commercial Point Facility | Fox Point Facility |
|---|---|---|
| Estimated Annual CSO Volume (from combined areas), mil.gal. | 38.0 | 22.6 |
| Estimated Annual Stormwater Volume (from separated areas), mil. gal. | 106.0 | 18.0 |
| Total Annual Volume, mil. gal. | 144.0 | 40.6 |

Since the analysis of the long-term rainfall record for this area indicated that storm events are distributed relatively evenly over the four seasons of the year, the total annual volume of CSO's and separate storm runoff that would require disinfection during the bathing season (i.e., the months of June, July, August and September) is estimated to be about 48 mil. gal. for Commercial Point, and 14 mil. gal. for Fox Point. As discussed in Chapter XI, assuming a 15 mg/l chlorine dosage, the hypochlorite solution would be applied at a rate of about 100 gallons per million gallons of waste stream treated. Thus, the total volume of 12% sodium hypochlorite solution required each year (i.e., each bathing season) is estimated to be about 5,000 gallons at Commercial Point and 1,500 gallons at Fox Point. The assumed chlorine dosage is considered a conservative estimate (high rate), and the actual dosage applied would be determined by pilot studies during the final design of each facility. The hypochlorite would be delivered in bulk to each facility by tank truck, and pumped from the truck into fiberglass reinforced plastic (FRP) tanks located within the superstructure. The tanks at each facility have been sized to store the entire annual volume of chemical required assuming one delivery at the start of the bathing season. However, two to three deliveries per bathing season is suggested since the disinfecting capability of hypochlorite is reduced over long periods of on-site storage (a 12% solution will lose about half of its original strength in about 120 days). The preliminary layout of the two facilities include an 8-ft. diameter by 14-ft. high FRP tank at Commercial Point (5,264 gallons), and a 6-ft. diameter by 8-ft. high FRP tank at Fox Point (1,696 gallons).

The solution would be applied using a variable-speed, chemical feed pump which would inject it into the waste stream through a perforated pipe diffuser, with the feed rate controlled by the rate of flow in the conduit. The contact time at each facility would be provided in the existing CSO conduit located downstream of the facility. It is estimated that under design storm conditions, the contact time at peak flow would be about 10 minutes for Commercial Point and about 17 minutes for Fox Point. The average contact time during the design storm is estimated at 17 minutes and 33 minutes, respectively, for the two facilities.

A chlorine residual analyzer would be provided in order to continually measure end of pipe residuals. This data will be used to determine the optimum hypochlorite feed rate that would result in an average end of pipe chlorine residual in the range of 1-2 mg/l. Such a residual would probably assure adequate coliform kill while also resulting in minimal adverse impacts to the receiving water. Since the facilities must be capable of handling extreme variations in waste stream flow rates due to various storm and tidal conditions, control of chlorine dosage rates would essentially be on an experimental basis during the initial start up period. Pilot plant studies at the time of final design could serve to reduce the degree of experimentation, however it is expected that actual operational experience would be required. As operational experience increases, and long-term hypochlorite feed rate vs. chlorine residual data is analyzed for several storm and tidal conditions, final operating policy would then be formulated.

Heating, Ventilation and Odor Control. Referring to Figure XII-26, the operator's room housing the electrical controls, the washroom and the utility room would be separated within the building by enclosures. Since their heating requirements are greater than the screening and disinfection equipment, separate heaters would be provided to avoid heating the entire building to the required higher temperatures. Heaters operating off of a separate thermostat would be provided in the main operations portion of the building. It is recommended that the hypochlorite tank and feed equipment be drained during the winter months to avoid potential freezing problems and to reduce winter heating requirements. The "left-over" chemical could be used at other MDC facilities such as the Cottage Farm Detention/Chlorination Facility, which applies this disinfectant year-round. It is suggested that a minimum ambient temperature of 40°F be maintained at each facility and increased, as required, during operation to increase operator comfort.

Odor control equipment has been included in the preliminary layout of each screening/disinfection facility. An air exchange unit would exhaust air from the CSO channel below the operating floor, pass it through activated carbon contained in plastic cells, and vent it through the roof of the structure. Fresh air would be drawn into the channel through floor grates. The exhaust air duct would be located along the interior wall and across the ceiling between the tee-beam webs to the odor control equipment located above the three separated rooms. A sill cock and hose would also be provided for washing down the screens between operations in order to reduce odors from accumulations on the screens.

It should be emphasized that during final design, space for odor control equipment should be provided, however, the actual need for odor control should be determined after the facility has been in operation for a period of time for reasons discussed earlier in this chapter (refer to the discussion of the South Boston storage/containment facility).

Superstructure. The buildings would have brick and concrete masonry walls with flat roofs of prestressed-concrete double-tee beam construction. The proposed bridge cranes would be suspended from separate steel beams supported on interior steel columns. For security purposes. there would be no windows, except perhaps for the electrical switchgear room and operator's office. There would be an exterior steel frame supporting the monorail extension (interlocking with the bridge crane trolley beam) to permit dumping of the screenings into a light dump truck. As shown on Figure XII-26, the electrical switchgear room and operator's office, the staff washroom, and the utility room (for buckets, maps, cleaning supplies, and spare parts) would be one-story rooms of concrete block construction partitioned off within the main superstructure. There would be a large interior window in the partition between the operator's office and the equipment space.

The building walls and roof would be insulated in conformance with current energy-conservation requirements.

Foundation Requirements and Construction Procedures. Similar to the South Boston Facilities, preliminary foundation requirements and construction procedures for each screening/disinfection facility, including the diversion conduit, were developed in conjunction with Haley and Aldrich, Inc. Test borings were drilled at each site (Figures XII-23 and XII-24) to obtain information on existing subsurface soil and ground water conditions. Results of Haley and Aldrich's analysis indicated that the Fox Point diversion conduit and screening/disinfection facility would be supported on medium capacity, pre-stressed concrete piles driven into the clay stratum approximately 90 to 100 feet, and designed to support the structural load in friction along the pile perimeter. For the Commercial Point diversion conduit and screening/ disinfection facility, the subsurface analyses indicated that the structures should be supported on treated timber end-bearing piles. The reports by Haley and Aldrich, which include discussions on subsurface explorations and conditions, preliminary design recommendation, and construction considerations for each screening/disinfection facility, are included in Appendix C.

Operation and Maintenance Requirements. The screening and disinfection processes at each facility would be controlled automatically by the methods described earlier in this chapter. The following functions would be sensed and recorded at each facility: the depth, rate, and total volume of flow in the existing CSO conduit upstream of the new bascule gate; the hypochlorite feed rate; and the end-of-pipe chlorine residual. The depth, and possibly, rate of flow in the screening channel and the position of the bascule gate would be telemetered to the continuously manned Columbus Park Headworks in order to monitor the status of the facility and to determine when flow through the facility has initiated. Each facility would be manned during a storm event to assure proper operation of the equipment. Upon completion of the storm, the screenings hoppers would be emptied, all equipment would be inspected, the screens washed down and the building secured. Daily inspection of each facility between storm events would also be performed. Estimated labor requirements for each facility are summarized as follows:

| Category | Civil Service Grade | Man-Hours per Week | Man-Hours per Year |
|---|---|---|---|
| Supervisory | 25-22 | 3.0 | 156.0 |
| Skilled Trades | 16-13 | 15.0 | 780.0 |
| Unskilled | 9-6 | 15.0 | 780.0 |
| Totals | | 33.0 | 1716.0 |

Energy requirements include the electricity to operate the hydraulic accumulator system (equipped with about a 5 horsepower motor), bar screen motors, chemical feed pumps, bridge crane and hoist motors, air compressor for the depth-of-flow bubbler system, heaters, odor control equipment and lighting. It is estimated that approximately 30,000 kilowatt-hours of electricity per year would be utilized at each facility.

Other operation and maintenance requirements include an estimated annual sodium hypochlorite requirement of 5000 gallons at the Commercial Point facility and 1,500 gallons at the Fox Point facility. The removal of screenings would occur after each operation, which is assumed to occur approximately once per week from each facility. An MDC-owned light dump truck similar to the one employed for this purpose at the Columbus Park headworks (the same vehicle could, most likely, be used), would be used to haul screenings from each facility as well as from the South Boston storage/containment facility to the Deer Island wastewater treatment plant for ultimate disposal. Miscellaneous equipment repair and maintenance would also be performed, as required, at each facility. A complete summary of the estimated annual costs associated with the operation and maintenance requirements of each facility is presented later in this chapter.

Estimated Costs. Estimated capital and annual operation and maintenance costs for each facility are summarized on Table XII-6. All costs represent 1979 price levels (ENR=2900).

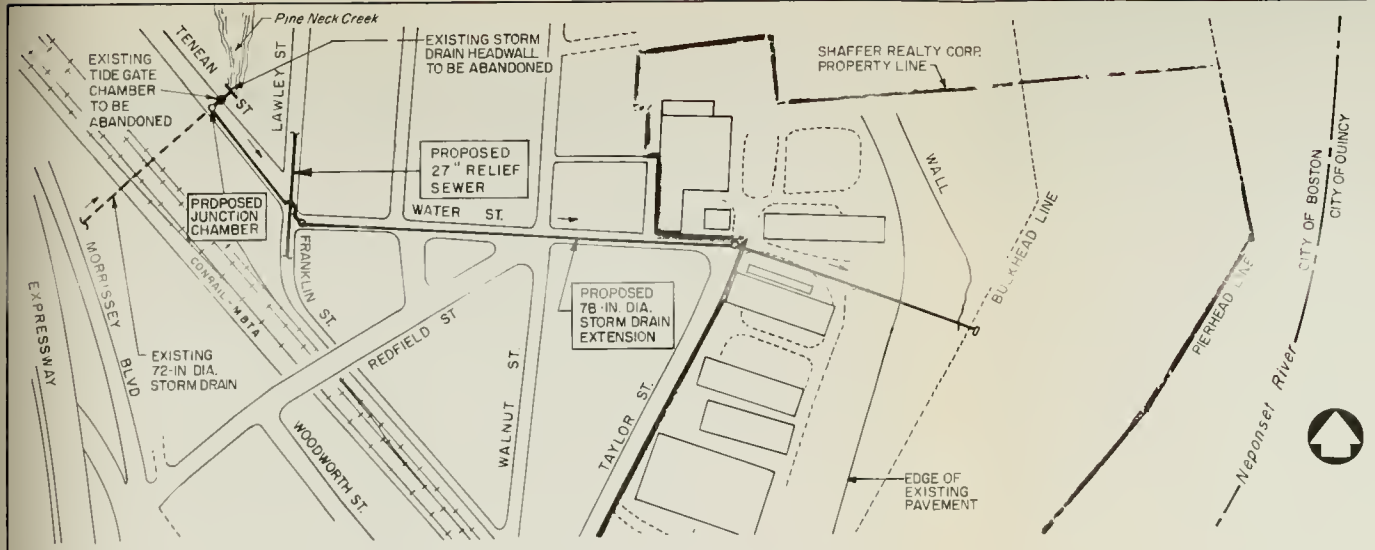
Pine Neck Creek Storm Drain Relocation. The need for diverting the storm drainage emanating from the Pine Neck Creek storm drain away from Tenean Beach was demonstrated in Chapter X and discussed further in Chapter XI. Three alternate routes for extending the existing 72-in. diameter storm drain to the Neponset River were investigated during the detailed evaluation phase. At that time, a route following Tenean Street, Franklin Street and an easement through private property to the Neponset River (Figure X-11 in Chapter X) was selected. The new conduit was shown in profile for this route on Figure XI-17.

General Description and Design Criteria. Further study of the alternate routes during the preliminary design phase indicates that a route in Tenean and Water Streets would be the most cost-effective, as well as the most feasible from a construction standpoint. From discussions with the current property owner, it appears that an easement could be obtained (through negotiation) for the downstream portion of the new conduit. The recommended route is shown on Figure XII-27.

TABLE XII-6 SUMMARY OF ESTIMATED COSTS FOR COMMERCIAL POINT AND FOX POINT SCREENING/ DISINFECTION FACILITIES

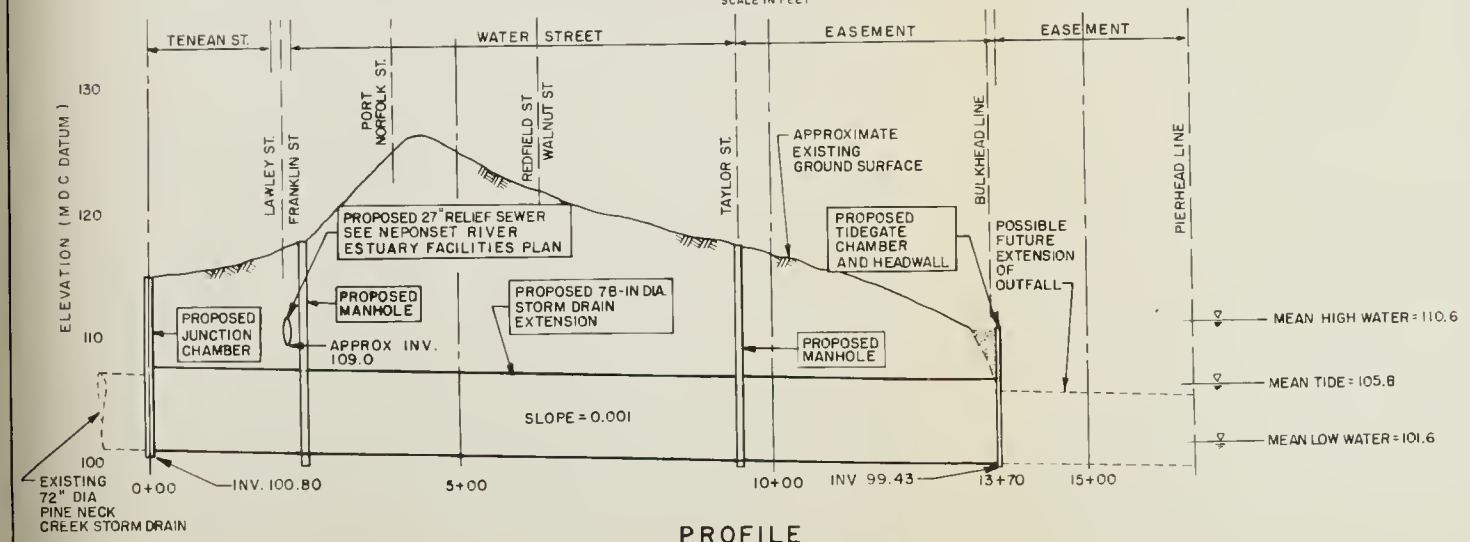
| ESTIMATED CAPITAL COSTS* | COMMERCIAL POINT FACILITY | FOX POINT FACILITY |
|---|---|---|
| Relocate 42" Storm Drain | $ - | $ 60,000 |
| Relocate Basketball Court | 15,000 | - |
| Diversion Conduit | | |
| Excavation & Sheeting | 200,000 | 145,000 |
| Piles | 45,000 | 35,000 |
| Concrete | 325,000 | 120,000 |
| Bascule Gate and Dry Pit | | |
| Excavation & Sheeting | 75,000 | 75,000 |
| Piles | 20,000 | 20,000 |
| Concrete | 120,000 | 120,000 |
| Mechanical Equipment & Controls | 170,000 | 170,000 |
| Screening/Disinfection Facility | | |
| Excavation & Sheeting | 115,000 | 115,000 |
| Piles | 25,000 | 25,000 |
| Concrete | 220,000 | 220,000 |
| Superstructure | 620,000 | 620,000 |
| Mechanical Equipment & Controls | 380,000 | 380,000 |
| SUBTOTAL | $2,330,000 | $2,105,000 |
| Engineering and Contingencies (30%) | 700,000 | 635,000 |
| TOTAL | $3,030,000 | $2,740,000 |
| Estimated Annual Operation and Maintenance Costs* | | |
| Labor | $ 12,000 | $ 12,000 |
| Power | 5,000 | 5,000 |
| Chemicals | 1,600 | 600 |
| Screenings Removal | 1,400 | 1,400 |
| Miscellaneous Equipment Repair and Maintenance | 2,000 | 2,000 |
| TOTAL | $ 22,000 | $ 21,000 |

*Estimated costs reflect June 1979 price levels (ENR=2900)




## PLAN

SCALE IN FEET 200 0 200

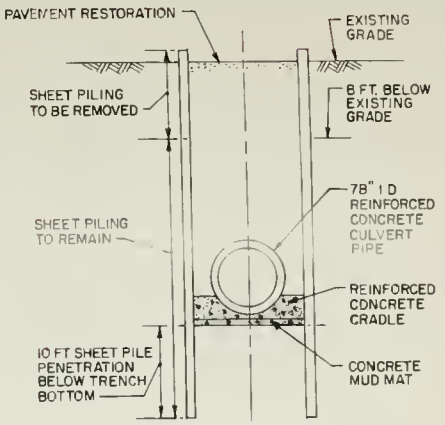



## PROFILE

HORIZONTAL SCALE IN FEET 200 0 200

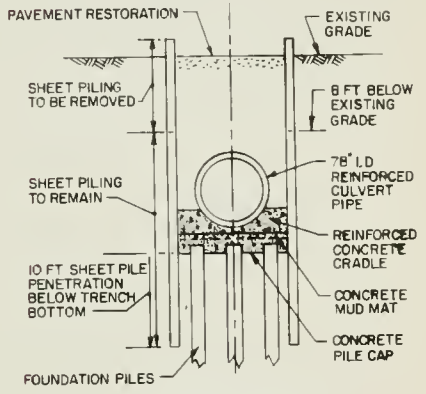
VERTICAL SCALE IN FEET 10 0 10

NOTE: EXISTING UTILITIES ARE NOT SHOWN


PAVEMENT RESTORATION
EXISTING GRADE
SHEET PILING TO BE REMOVED
8 FT. BELOW EXISTING GRADE
78" I D REINFORCED CONCRETE CULVERT PIPE
SHEET PILING TO REMAIN
REINFORCED CONCRETE CRADLE
10 FT SHEET PILE PENETRATION BELOW TRENCH BOTTOM
CONCRETE MUD MAT

## TRENCH SECTION PIPE ON CONCRETE CRADLE

(STA. 2+50 TO STA 11+60)




## TRENCH SECTION PIPE ON PILES

(STA 0+00 TO STA. 2+50 & STA 11+60 TO STA 13+70)

COMMONWEALTH OF MASSACHUSETTS

METROPOLITAN DISTRICT COMMISSION

COMBINED SEWER OVERFLOW PROJECT

DORCHESTER BAY AREA FACILITIES PLAN

PINE NECK CREEK STORM DRAIN RELOCATION

FIG. XII-27

As proposed, the existing 72-in. diameter conduit would be intercepted by the new conduit immediately above the existing tide gate chamber located on the north side of Tenean Street. The new conduit would be 78-in. in diameter and constructed at a slope of 0.001 ft/ft. This size was determined by analyzing the hydraulics of the existing storm drain system under a high tide condition, and providing equal flow capacity for the new storm drain extension. It is estimated that under a high tide condition, the drainage system, with the proposed extension, could surcharge to provide a maximum hydraulic grade line slope of 0.0015 ft/ft resulting in a flowing full capacity of about 135 mgd for the new 78-in. conduit. This capacity equals the capacity of the existing drainage system under the same tidal condition, and is adequate to convey the estimated peak rate of runoff from a 15-year frequency storm event from its 118-acre tributary area (estimated to be approximately 115 mgd).

Under the recommended route, the new, 78-in. conduit would follow Tenean Street to Lawley Street; in Lawley Street to Franklin Street; then easterly in Water Street to Taylor Street; and then to the Neponset River bulkhead line in an easement through property presently owned by the Shaffer Realty Corporation, an affiliate of Shaffer Paper Fibres, Inc. As shown on Figure XII-27, the Shaffer parcel extends to the pierhead line on the west side of the Neponset River. The total length of the new drain extension along this route would be about 1400 feet, and the outfall would terminate in a new tidegate chamber (to be constructed on piles) just beyond the bulkhead line. In the event the Shaffer property were further developed in the future by decking on piles beyond the bulkhead line, a future extension of the proposed 78-inch outfall beyond the bulkhead line (shown as a future possiblity on the profile on Figure XII-27) may be necessary. The present cost estimate does not include the possible future extension.

Foundation Requirements and Construction Procedures. Similar to the South Boston facilities and the Dorchester screening/disinfection facilities, preliminary foundation requirements and construction procedures were developed in conjunction with Haley and Aldrich, Inc., utilizing available subsurface information along the proposed alignment. The foundation requirements recommended for preliminary design include pile support between Stations 0+0 and 2+50 and between Stations 11+60 and 13+70, where it is estimated that the thickness of organic soils is greater than 5 feet. From Station 2+50 to 11+60, available subsurface information indicates earth support is possible.

Due to the fluctuating groundwater levels resulting from tidal movements, it is recommended that the entire length of the new conduit be constructed on a concrete cradle to assure adequate bedding. Typical cross-sections of the conduit showing the two methods of support are shown on Figure XII-27. The report by Haley and Aldrich, which includes discussions on available subsurface information, assumed subsurface conditions, preliminary design recommendations and construction considerations, is included in Appendix C.

Estimated Costs. The total estimated capital cost of the Pine Neck Creek storm drain relocation is summarized as follows (1979 Price Levels, ENR 2900):

| | |
|---|---|
| Sheeting, Excavation, Dewatering and Backfill | $1,035,000 |
| Concrete | 55,000 |
| Piles | 140,000 |
| Pipe | 400,000 |
| Structures | 170,000 |
| Subtotal | $1,800,000 |
| Engineering and Contingencies (30%) | 540,000 |
| Total | $2,340,000 |

## Non-Structural Recommendations

Non-structural recommendations, as presented in Chapter XI, include the dry weather overflow abatement program and best management practices. The primary purpose of these recommendations is to reduce the amount of pollution entering Dorchester Bay from improper operation of the sewerage system.

Dry Weather Overflow Abatement Program. The elimination of dry weather overflows (DWO's) is imperative if Dorchester Bay is to meet SB water quality standards. The first step in eliminating DWO's is the identification of the sources. System operational problems and improper sanitary connections to storm drains need to be identified, and corrective measures implemented. The purpose of the Dry Weather Overflow Abatement Program is to identify potential DWO sources, and recommend corrective measures.

Work tasks associated with the Dry Weather Overflow Abatement Program were outlined previously in Chapter XI. In summary, the program calls for a physical survey of the existing sewerage and storm drainage system including dye testing to identify existing unknown dry weather flow cross connections. The total estimated labor required to implement the program is 1,560 person-days, 80 percent of which would be for field work, and the remaining for record keeping and defining the recommended corrective measures. Two field inspection vehicles would be required, complete with system access equipment. The estimated duration of the program is one year.

The total estimated cost of the program at 1979 price levels (ENR 2900) is $100,000, assuming that the City of Boston (i.e., Boston Water and Sewer Commission) performs the recommended program. A breakdown of the estimated cost is presented below.

| | |
|---|---|
| LABOR - Massachusetts Civil Service Grade Classificatons | |
| 1 - Supervisor (Grade 25-22) - Administrative | $23,500 |
| 1 - Engineer (Grade 21-17) | 19,000 |
| 2 - Skilled (Grade 16-13) | 30,000 |
| 2 - Unskilled (Grade 9-6) | 21,000 |
| Subtotal | $93,500 |
| VEHICLE OPERATION AND MAINTENANCE | 65,000 |
| Total | $100,000 |

Best Management Practices. As discussed in Chapter XI, the following best management practices, required to assure proper long-term system operation, are recommended:

- o Regulator maintenance
- o Catch basin cleaning and maintenance
- o Sewer cleaning
- o Tide gate repair and maintenance

This continually on-going maintenance program would be initiated upon completion of the Dry Weather Overflow Abatement Program. It would require the purchase of one maintenance vehicle, most likely a light dump truck, and the necessary cleaning equipment and maintenance and repair tools. Capital costs for these expenditures are estimated at $25,000 at 1979 price levels (ENR 2900).

It is recommended that one, three-man crew be assigned full time to the Dorchester Bay study area for regulator maintenance, sewer cleaning and tide gate repair and maintenance. Sewer cleaning would be performed, as required, and would require the use of existing Boston Water and Sewer Commission equipment (i.e., sewer jet rodder). Inspection of the 34 regulators and 28 tide gate chambers within the study area would be performed weekly, with maintenance and repair occurring as required. The annual labor cost is estimated to be $46,000, with the annual vehicle operation and maintenance cost estimated at $4,000 (1979 price levels). Since there are approximately 3,000 catch basins within the study area, it is recommended that the City of Boston's current practice of contracting with an outside concern for catch basin cleaning be continued, however increased to allow each basin to be cleaned once per year. Currently, each basin is

cleaned on the average of once every two to three years. The estimated annual cost for catch basin cleaning is $150,000 at 1979 price levels. A summary of the estimated costs for implementation of the best management practices at 1979 price levels (ENR 2900) is presented below.

CAPITAL COSTS

| | | |
|---|---|---|
| Maintenance Vehicle | | $ 21,000. |
| Equipment | | 4,000. |
| | Total Capital | $ 25,000. |

ANNUAL COSTS

| | |
|---|---|
| Labor - Massachusetts Civil Service Grade Classifications | |
| 1 - Supervisor (Grade 25-22) - part time | $ 10,000. |
| 1 - Skilled (Grade 16-13) | 15,000. |
| 2 - Unskilled (Grade 9-6) | 21,000. |
| Vehicle Operation and Maintenance | 4,000. |
| Catch Basin Cleaning (Contractural Basis) | 150,000. |
| Total Annual | $200,000. |

It is also recommended that the City of Boston's street cleaning policy be maintained, since street cleaning serves to remove nutrient and heavy metals that typically wash-off during storm events into the combined system.

## Miscellaneous Recommendations

As mentioned at the beginning of this chapter, three additional items are included as part of the total recommended plan and are considered necessary to assure proper system operation. These items consist of the cleaning of CSO conduits, dredging at the CSO outlets, and recommendations involving the landfill located at CSO outlet BOS-090. A discussion of each follows.

Cleaning of CSO Conduits. Excessive sediment deposition in CSO conduits reduces flow conveyance capacity and, during large storm events, could result in the flooding of upstream regulators. Accordingly, it is recommended that all CSO conduits be inspected and excessive sediment deposits be removed.

Based on field investigations, an average sediment depth of 2 ft was found in the conduit leading to BSO-090 between Christopher Street and the outlet (Figure C, appended). The cause of deposition is, most likely, due to the low conduit flow velocities resulting from the hydraulic restriction imposed by the landfill located at the outlet. This situation was discussed in Chapter IV, and recommendations involving the landfill are presented subsequently. About 1 to 2 ft of sediment was found in the downstream section of the conduit leading to BOS-089, from the tide gate chamber near Morrissey Boulevard to the outlet at Fox Point. No sediment deposition was found in any of the South Boston CSO conduits inspected; however, further investigations should be performed at the time CSO conduit cleaning in the Dorchester area is undertaken.

Based on the extent of deposition found during field investigations for this study, the estimated capital cost for its removal is $300,000. This cost is based on removing 2,500 cu. yds at $120 per cu. yd, which was the unit cost for sediment removal during the cleaning of the Dorchester interceptor (Chapter IV).

Dredging. Dredging at the CSO outlets is recommended in order to remove objectionable sediment deposits and to assure an unobstructed flow path during CSO discharge events. During field investigations, sediment deposits were found at outlets BOS-089 (Fox Point) and BOS-090 (Commercial Point). The deposition at the Fox Point outlet is located in an area where boats from the Savin Hill Yacht Club are moored and members of the yacht club have visually observed the deposits at low tide. During storm events and/or extensive wave action (i.e., high winds), it is likely that a portion of this sediment may become re-suspended, thus resulting in displeasing visual and aesthetic effects. At the Commercial Point outlet (BOS-090), the current landfill creates a "pool" containing objectionable sludge deposits which serve as an insect breeding ground during summer months, and it also causes severe odors, especially during low tide conditions. This site is located adjacent to the Old Colony Yacht Club. Sediment deposits at the South Boston CSO outlets were not observed during field investigations since all outlets are submerged even during low tide conditions. It is recommended, however, that the extent of sediment deposition at the South Boston outlets be determined, and the sediment be removed at the outlets where the deposits significantly obstruct the flow path.

Discussions with area dredging contractors indicated that the cost of sediment removal would be about $15 per cu. yd, which is considered a conservative estimate. If it is assumed that approximately 2,000 cu. yds of sediment is removed at the outfalls, the total estimated cost of dredging would be in the order of $30,000. The dredged material would, most likely, be barged to the foul dump area currently located approximately 15 miles offshore of Manchester (Mass.) Harbor. A dumping permit from the Army Corps of Engineers would be required and, if obtained, would satisfy all federal regulations pertaining to ocean dumping (see Chapter XIII). According to officials from the permits branch of the Corps, a bio-assay and bio-accumulation test would be required on the dredged material prior to issuing the permit. The cost of these tests would be about $10,000, which would increase the total estimated dredging cost to about $40,000.

Landfill at CSO Outlet BOS-090. A description of the existing landfill located at CSO outlet BOS-090 near Commercial Point (Figure C, appended) was presented in Chapter IV. In that description, the problems relating to the hydraulic restriction to flow emanating from the CSO conduit due to the landfill were also discussed. In addition, the configuration of the landfill resulted in the formation of an extensive sludge deposits at the outlet, since flow ponds in this area prior to discharging through a 36-in. pipe which extends through the pool's earthen dike. This sludge deposit would be removed under the recommended dredging program, however, elimination of this flow restriction is required to prevent further sludge build-up. It would also serve to increase flow capacity and velocity in the CSO conduit and, accordingly, reduce the extent of sediment deposition.

Norwood Engineering Company, Inc. and Carr Research Laboratory, Inc., performed an engineering investigation on alternatives regarding potential recreational uses or elimination of the current landfill for the Boston Conservation Commission. Their draft report recommended the removal of the dike and construction of a drainage culvert using rip-rap. The report also recommended that the landfill remain and be utilized for recreational purposes. Several alternative recreational plans were included in their report.

It is recommended that the earthen dike which restricts the discharge of flow from the CSO conduit be eliminated, in conformance with the above report. It is also suggested that rip-rap be installed forming a drainage channel from the end of the CSO conduit, through the earthen dike (which would have been removed) and terminated at a point beyond the landfill about 100-ft from the dike. The rip-rap would provide a stable sub-base and would facilitate flow conveyance to beyond the landfill. A conceptualization of this recommendation is shown on Figure C, appended. The estimated capital cost is $20,000.

Residual Waste Management

Implementation of the recommended plan would result in residual wastes generated by the South Boston storage/containment facility and by the two Dorchester screening/disinfection facilities. In addition, wastes consisting of sediment from catch basins, sewers and CSO conduits, and spoils from the recommended dredging at CSO outlets would also be generated.

As presently conceived, the screenings generated at the South Boston storage/containment facility and at the two Dorchester screening/disinfection facilities would be hauled, most likely, to the Deer Island Wastewater Treatment Plant (WWTP) for disposal by burial at the existing landfill. This is the current method of disposal of the screenings (and grit) generated at the Columbus Park headworks, as well as from the two other headworks within the Deer Island WWTP tributary system (Ward Street and Chelsea Creek). If the capacity of this landfill is exceeded in the future and a new landfill is secured, disposal of solid wastes generated by implementation of the recommended plan would occur at the new landfill. The estimated quantities of screenings generated at each proposed facility are summarized as follows:

| | Total Volume of Screenings, cu. ft | |
|---|---|---|
| | Average Weekly | Average Annual |
| South Boston storage/containment facility | 5 | 260 |
| Commercial Point screening/disinfection facility | 25 | 1,300 |
| Fox Point screening/disinfection facility | 7 | 380 |
| Total | 37 | 1,940 |
| Current volumes generated at the Columbus Park headworks | 270 | 14,000 |
| % increase | 14 | 14 |

The sediment removed from catch basins, sewers and CSO conduits would be disposed of by burial at the City of Boston landfill which is also utilized by the BWSC for disposal of these spoils. The landfill currently used is located on Gardner Street in Roxbury; however, use of this site is scheduled to be discontinued this year (1980) with the new site yet to be determined. As discussed earlier in this chapter, it is estimated that in the order of 2,500 cu. yds of sediment would be removed from the CSO conduits. Assuming an average of approximately 1 cu ft of sediment is removed annually from each of the 3,000 catch basins with the study area, the recommended annual catch basin cleaning program may generate in the order of 3,000 cu. ft (110 yds) of sediment per year. Sewer cleaning, when required, would also generate spoils requiring disposal.

The South Boston storage/containment (SBS/C) facility involves pumpback of CSO's from the storage tank to the Columbus Park headworks. This practice would result in increases in the volume of flow, total suspended solids (TSS) load and volume of grit at the headworks. The increases in flow and TSS would also impact on the Deer Island WWTP, however, grit would be removed at the headworks by the existing grit channels. These impacts are summarized as follows:

Volume of Flow, Mil. gal.

| | |
|---|---|
| Average annual at headworks | 25,000 |
| Average annual from SBS/C facility to headworks | 130 |
| % increase | 0.5 |

Volume of Grit, cu. ft

| | |
|---|---|
| Average annual removed at headworks | 5,800 |
| Average annual from SBS/C facility to headworks | 800 |
| % increase | 14.0 |

TSS Load, Lbs

| | |
|---|---|
| Average annual through headworks | 27,100,000 |
| Average annual from SBS/C facility to headworks | 55,000 |
| % increase | 0.2 |

The impact of the residual wastes generated annually by implementation of the Dorchester Bay Area's recommended plan that would require disposal by burial at the existing or new Deer Island WWTP landfill and at a City of Boston landfill is summarized as follows:

| | Average Annual Volumes, cubic feet | | | |
|---|---|---|---|---|
| | Screenings | Grit | Sediment from System Maint. | Total |
| Current quantities generated by Columbus Park headworks | 14,000 | 5,800 | - | 19,800 |
| Estimated quantities generated by current system maintenance (from catch basin cleaning assuming each basin cleaned once every 2.5 yrs). | - | - | 1,200 | 1,200 |
| Total estimated present quantities | 14,000 | 5,800 | 1,200 | 21,000 |
| Estimated quantities generated by implementation of the recommended plan | 1,940 | 800 | 5,000* | 7,740 |
| Total | 15,940 | 6,600 | 6,200 | 28,740 |
| Estimated percent increase | 14 | 14 | 417 | 37 |

In addition to the annual quantities presented above, the estimated 2,500 cu. yds (67,500 cu. ft) of sediment, generated by cleaning of the CSO conduits, would also require disposal at the City of Boston landfill. This would not be an annual maintenance item, but a single cleaning operation.

As discussed earlier in this chapter, disposal of dredging spoils would, most likely, be accomplished by barging to the off-shore foul dump area. The quantity of spoils is preliminary estimated at about 2,000 cubic yards. Permit requirements and estimated costs were discussed in the section on dredging earlier in this chapter.

In summary, it is estimated that approximately 1,940 cu. ft of screenings (i.e., floatables), 800 cu. ft of grit, and 55,000 lbs of TSS would be prevented from entering Dorchester Bay each year by implementation of the recommended plan. In addition, approximately 5,000 cu. ft of sediment is expected to be removed annually from the sewerage system within the Dorchester Bay study area. Additional estimated quantities removed from one-time operations include 2,500 cu. yds (67,500 cu. ft) of sediment from cleaning CSO conduits, and 2,000 cu. yds (54,000 cu. ft) of dredging spoils removed at CSO outlets. It should be emphasized that the annual quantities

*Includes 3,000 cu. ft from the catch basin cleaning program and an allowance of 2,000 cu. ft for sewer cleaning.

expected to be removed are based on expected average annual flowrates and volumes. Due to the variability of pollution loads in urban stormwater runoff, these quantities could deviate from those presented herein by as much as 20 to 30 percent.

## Impact of Recommended Plan on Dorchester Bay's Water Quality

General. The impacts of CSO's on Dorchester Bay were estimated on both a long-term and short-term basis. The existing and future no-action situations were presented in Chapter IX. A comparison of impacts resulting from these situations with those of the Recommended Plan are discussed herein.

Shown on Table XII-7 is a comparison of data for the existing situation and recommended plan. For the South Boston area, the number of total coliform violation events per year decreased 93 percent (56 to 4 events), and the number of floatables violation events 100 percent (56 to 0), with implementation of the recommended plan. For Dorchester, the percent reductions in the number of annual violations were 65 for total coliforms, and on a long-term basis, 100 for floatables. For the overall study area, the total pseudo-pounds of total coliforms per day entering the bay on a long-term basis would be reduced approximately 90 percent with implementation of the recommended plan. CSO-related pseudo-pounds would be reduced by about 70 percent; storm drainage by 80 percent; and it is assumed that dry weather overflows would be eliminated completely. The storm drainage percent reduction is due to the relocation of the Pine Neck Creek separate storm system outlet to the Neponset River. Accordingly, although the study area's pollutant load is reduced, the overall harbor's load from this source is not, since no treatment is provided.

The long-term impacts were estimated by the Harbor Modeling Consultant (HMC) using the output from SEMSTORM in the form of exceedence curves developed from the 29-year period of record. The short-term impacts were estimated by the HMC using time-variable modeling, and by the Dorchester Bay area consultant based on an evaluation of near-field dispersion and dilution of the CSO wastewater. A description of the harbor model, including the methodology and criteria used to determine receiving water impacts, is presented in Chapter VIII.

Discussed in Volume II - Chapter X are the environmental consequences of the recommended plan's implementation on the water quality of Dorchester Bay.

Long-Term Assessment. The HMC quantified the effect of intermittent runoff mass discharges on the water quality of Boston Harbor, including Dorchester Bay. Because of the random nature of the rainfall and runoff processes, the analysis of the receiving water response was made within a statistical framework. Daily concentrations are tabulated in time sequence, and percentile concentrations derived: $C_{80}$ - 80th percentile concentration which is exceeded 20 percent of the time. Further, the geometric mean ($C_{50}$) is also presented.

TABLE XII-7 COMPARISON OF DATA FOR EXISTING SITUATION AND RECOMMENDED PLAN

| | South Boston Area[6] | | Dorchester Area | | Total Study Area[1] | |
|---|---|---|---|---|---|---|
| | Existing Situation | Recommended Plan | Existing Situation | Recommended Plan | Existing Situation | Recommended Plan |
| Number of Overflow Events per Year | 56 | 4 | 52 | 24 | - | - |
| Number of Violation Events per Year | | | | | | |
| o Total Coliform[3] | 56 | 4 | 52 | 18[4] | - | - |
| o Floatables[3] | 56 | 0 | 52 | <1 | - | - |
| Pseudo Pounds[5] of Total Coliforms per day, $10^8$ | | | | | | |
| o CSO | | | | | 0.66 | 0.20 |
| o Storm Drainage | | | | | 0.20 | 0.04 |
| o DWO[2] | | | | | 1.44 | 0.00 |
| o Total | | | | | 2.30 | 0.24 |



Notes

1Average for total study area

[2]Including dry weather overflows identified during field program

[3]Parameter violations of SB standards - see Chapter IX for discussion.

[4]Assumes disinfection of CSOs at the Commercial Point and Fox Point screening facilities approximately six times per year during the bathing season.

[5]Pseudo pounds per day = Q (mgd) x concentration (No./100 ml) x 8 .34.

[6]Tributary to Old Harbor area only, and does not include BOS-080 to Reserved Channel. See text for discussion.

Shown in Table XII-8 are the total coliform results calculated by the harbor model for the existing situation and the recommended plan. Figures XII-28 and XII-29 show the results in a graphical format. The total coliform standard is compared with the calculated values for the 50th and 80th percentile concentrations in harbor model segment numbers corresponding to the five beach locations, as shown previously on Figure IX-1. Under the existing situation, Malibu Beach and Tenean Beach do not meet the standards at the 50th percentile concentration, and at the 80th percentile, these same beaches, along with Carson Beach and "L" Street Beach have unacceptable calculated values. As discussed in Chapter IX, the concentrations shown are for the centroid of the harbor model segment, and not at the beach's swimming area.

With implementation of the recommended Dry Weather Overflow Abatement Program and the system maintenance program, dry weather overflows will mitigate. The receiving water quality benefits associated with the compelte removal of dry weather overflows are shown in Table XII-8. Dry weather overflows have a substantial impact on the quality of Dorchester Bay waters, as illustrated by the fact that all areas, except for Tenean Beach, will meet the long-term standards if dry weather overflows are removed.

Implementation of the recommended plan further improves the Bay's water quality, to the extent that all locations shown in Table XII-8 meet the standards. The water quality in the Tenean Beach area is improved substantially for the following reasons:

1. The number of overflows per year from CSO outlet BOS-090 (Commercial Point) will decrease from approximately 52 to 24;

2. About 25 percent of the recommended plan's overflows occurring at BOS-090 on a long-term basis (i.,e., four-month bathing season) will be disinfected; and

3. The total coliform load form the Pine Neck Creek separate storm drain system will be diverted to the Neponset River under the recommended plan.

Referring to Table XII-8, except for the Tenean Beach area, the recommended plan is not required for the other locations to meet the long-term coliform standards. Accordingly, the recommended plan's major benefits are associated with the short-term coliform situation, and the removal of floatables. As discussed in Chapter XI, the recommended plan's major benefits are: (1) the reduction in the number of violation events during the summer months, thus increasing the use of the study area's bathing beaches, and (2) improving the aesthetics of the bay since CSO floatables would be eliminated.

TABLE XII-8 CALCULATED LONG TERM TOTAL COLIFORM RESULTS FOR EXISTING SITUATION AND THE RECOMMENDED PLAN[1]

| Location | Harbor Segment No. | Total Coliform Standard | | Existing Situation | | Existing Situtation Only DWO Removed[2] | | Recommended Plan | |
|---|---|---|---|---|---|---|---|---|---|
| | | 50th% | 80th% | 50th% | 80th% | 50th% | 80th% | 50th% | 80th% |
| Pleasure Bay | 29 | 700 | 1,000 | 100 | 300 | <10 | <10 | <10 | <10 |
| Carson Beach | 24 | 700 | 1,000 | 600 | 1,700 | 70 | 540 | <10 | 50 |
| "L" Street Beach | 24 | 700 | 1,000 | 700 | 1,800 | 80 | 470 | <10 | 30 |
| Malibu Beach | 12 | 700 | 1,000 | 1,700 | 4,500 | 140 | 820 | 20 | 140 |
| Tenean Beach | 213 | 700 | 1,000 | 5,200 | 14,300 | 390 | 2,200 | 80 | 400 |



Notes:

1 From tables 2a and 2b of Hydrosciences' March 7, 1980 "Boston Combined Sewer Overflow Study - Statistical Model Results for the Recommended Plan".

2 With dry weather overflows (DWO) identified during field program eliminated.

3 Number per 100 ml.

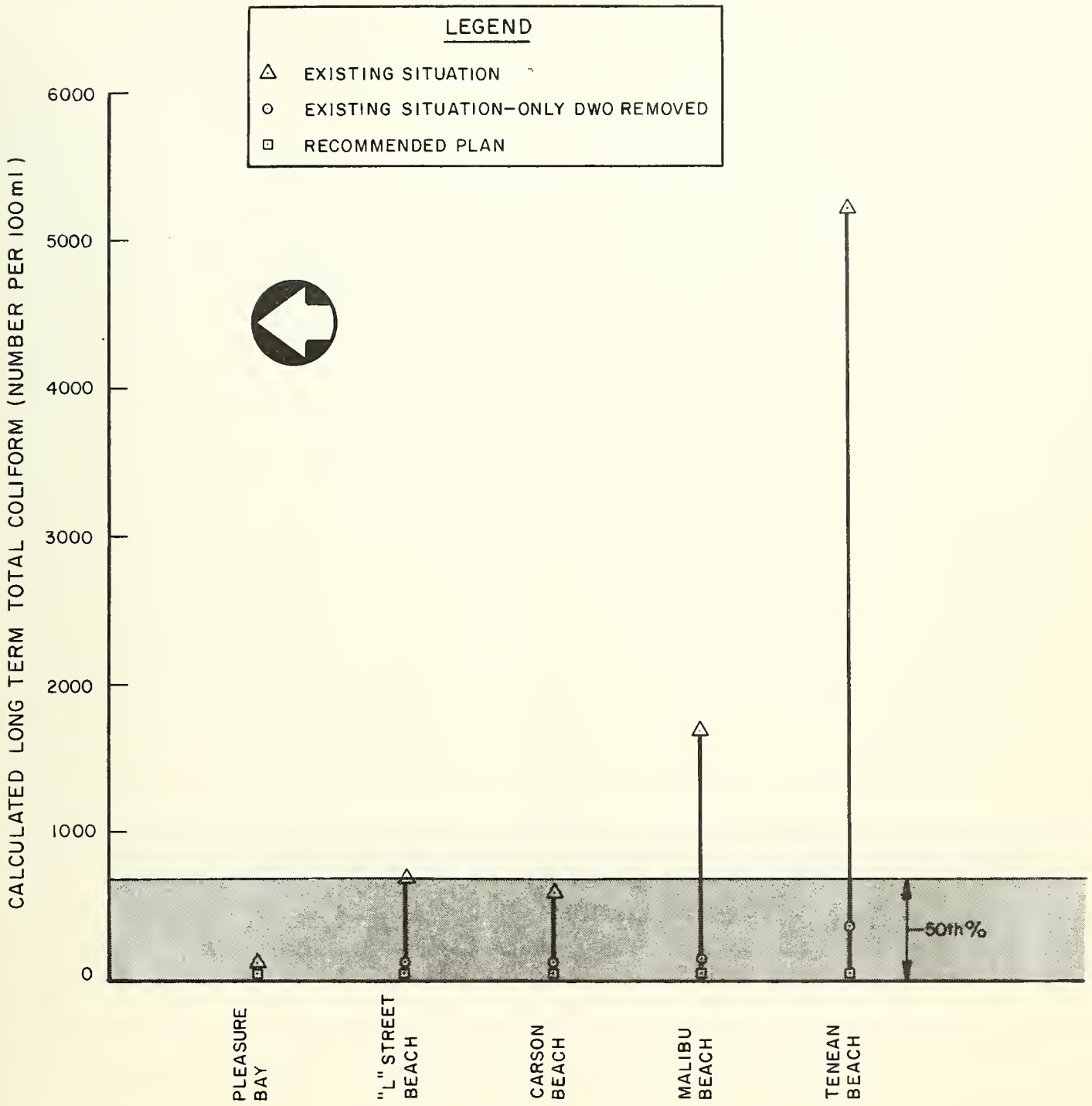



FIG. XII-28 CALCULATED LONG TERM TOTAL COLIFORM 50th% RESULTS - EXISTING SITUATION AND RECOMMENDED PLAN

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METROPOLITAN DISTRICT COMMISSION
COMBINED SEWER OVERFLOW PROJECT
DORCHESTER BAY AREA FACILITIES PLAN

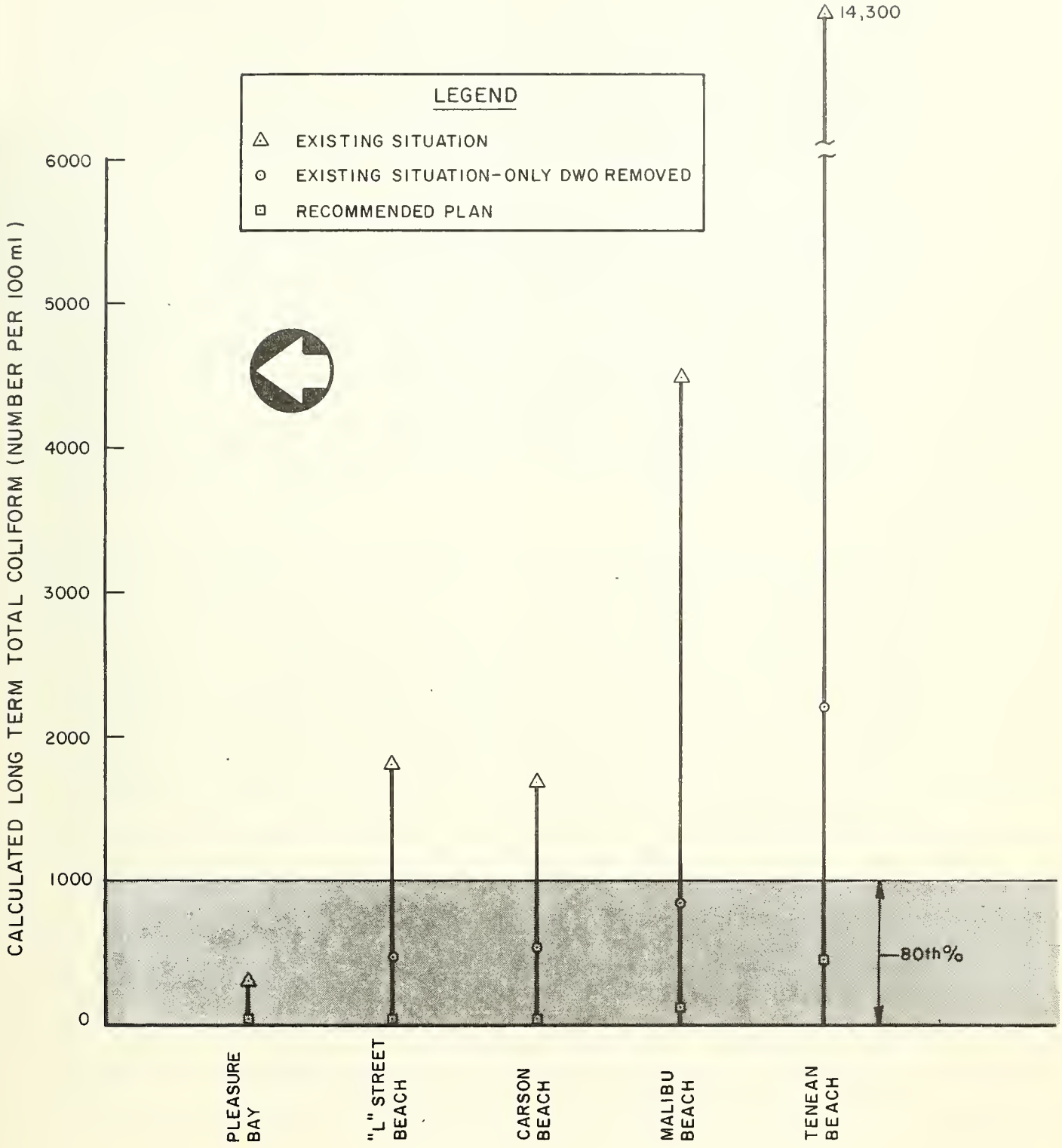


FIG. XII-29 CALCULATED LONG TERM TOTAL COLIFORM 80th% RESULTS-EXISTING SITUATION AND RECOMMENDED PLAN

COMMONWEALTH OF MASSACHUSETTS
METROPOLITAN DISTRICT COMMISSION
COMBINED SEWER OVERFLOW PROJECT
DORCHESTER BAY AREA FACILITIES PLAN

Short-Term Assessment. The purpose of assessing the situation on a short-term basis is to attempt to identify the impact of CSO's on the study area beaches immediately following an overflow event. The HMC utilized a time variable water quality model to estimate the impacts at certain locations within the bay for the design storm events. The Dorchester Bay area consultant supplemented this information by evaluating the physical and biological factors which affect the dilution and dispersion of the overflow wastewater in the vicinity of the outlets, and how the overflows affect the swimming beaches.

Described in Chapter VIII are the basic assumptions and criteria used for the HMC's time-variable water quality model. In summary, for the existing situation and the three design storm events during the summer season, the HMC estimated the total coliform concentrations in the bay. Dry weather overflow was not included in the runs, and it was assumed that a 15 MPH northeast onshore wind prevails constantly across the study areas. Shown on Fig. XII-30 are the locations of the modeling grids used to monitor the short-term total coliform concentrations for the Dorchester Bay study area. It should be noted that Segment 20, 2-"Mouth of Neponset River", reflects the resulting water quality for both the Dorchester Bay study area (i.e., BOS-089, BOS-090, and storm drainage) and the Neponset River study area loads.

Presented in Table XII-9 are the peak total coliform concentrations at the monitored segments for the three design storm events under the existing situation and the recommended plan. As discussed previously, the South Boston storage/containment facility is designed for the 3-month, 4-hour storm event. For storm events greater than this, overflows will occur via the existing outlets. The results shown in Table XII-9 reflect the quality improvements to the Carson Beach area waters with implementation of the recommended South Boston improvements. Although the 1-year, 6-hour storm exceeds the design capacity of the proposed storage/containment facility, the area's water quality is improved during this storm event, because CSO volumes up to those generated from the 3-month, 4-hour storm will be stored during the wet weather period. As shown in Table XII-10, the recommended plan reduces the calculated peak total coliform concentrations by 60 percent during the 1-year, 6-hour storm events, 98 percent for the 3-month, 4-hour event, and 99 percent for the 1-month, 4-hour event.

In Segment 20, 2 (Mouth of Neponset River), the water quality improvements associated with the installation of the study area's recommended Fox Point and Commercial Point screening/disinfection facilities are not as noticeable as the South Boston improvements. The results indicate that a 70 percent reduction (Table XII-10) in the peak total coliform concentrations is expected in this modeling segment. Greater reductions are not realized because of the amounts of Neponset River pollutant loads entering the estuary area. Calculated peak coliform concentrations exceed the recommended 1000 MPN/100 ml limit during all storm events.

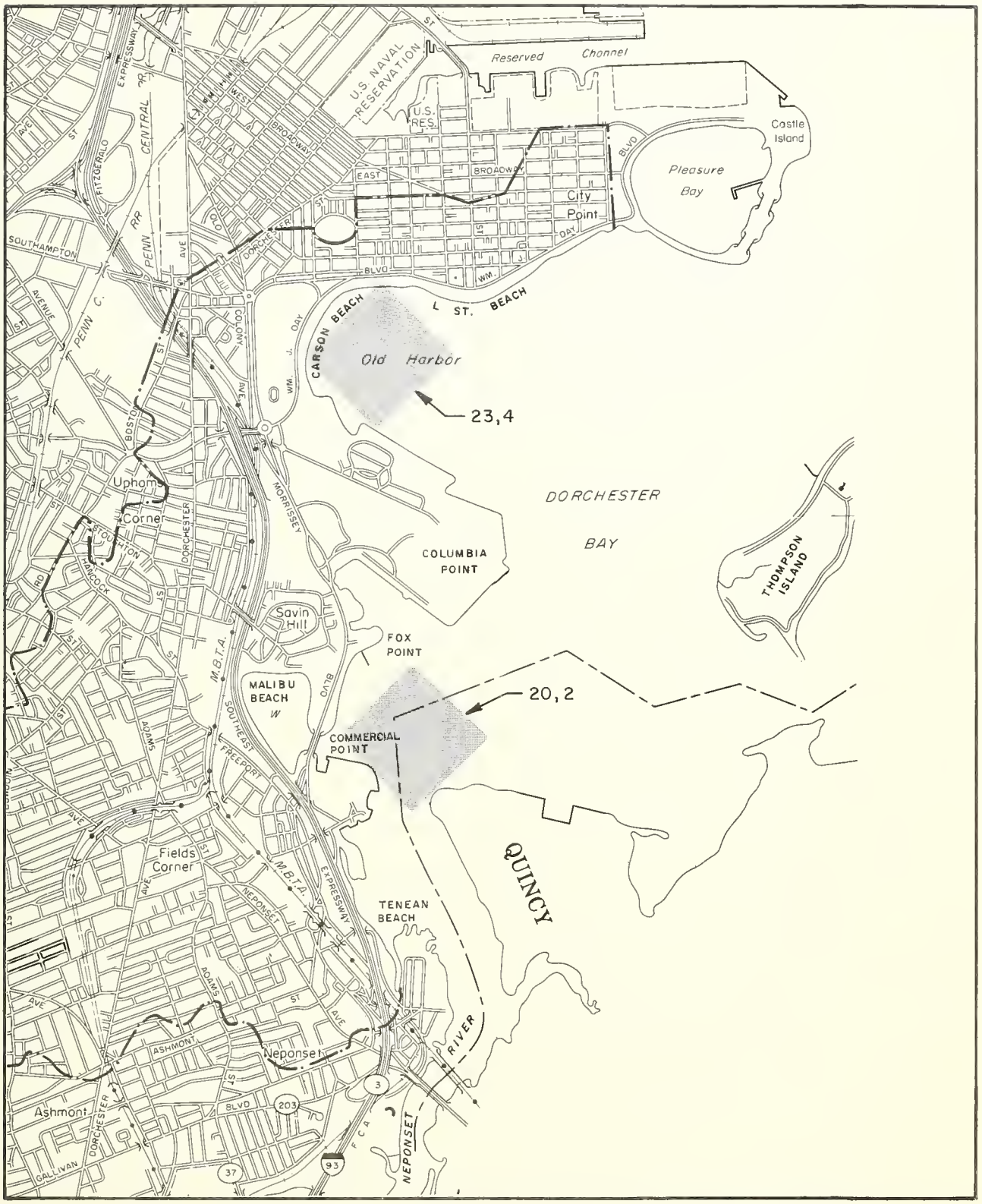


FIG. XII-30 TIME VARIABLE MODELING GRIDS

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DORCHESTER BAY AREA FACILITIES PLAN

TABLE XII-9 TIME-VARIABLE WATER QUALITY MODELING CALCULATED RESULTS - PEAK COLIFORM CONCENTRATIONS DURING DESIGN STORM EVENTS FOR THE EXISTING SITUATION AND RECOMMENDED PLAN [1]

| | Peak Total Coliform Concentrations, MPN/100 ml | |
|---|---|---|
| | Carson Beach Segment 23, 4 | Mouth of Neponset River Segment 20, 2 |
| 1-Year, 6-Hour Storm | | |
| o Existing Situation[2] | 65,000 | 490,000 |
| o Recommended Plan | 25,000 | 140,000 |
| 3-Month, 4-Hour Storm | | |
| o Existing Situation[2] | 15,000 | 230,000 |
| o Recommended Plan | 250 | 68,000 |
| 1-Month, 4-Hour Storm | | |
| o Existing Situation[2] | 15,000 | 100,000 |
| o Recommended Plan | 100 | 30,000 |

Notes:

[1] From Table 2 of Hydroscience's February 29, 1980 time-variable modeling work submittal. See text discussion on criteria used.

[2] Not including dry weather overflows.

TABLE XII-10 TIME-VARIABLE WATER QUALITY MODELING CALCULATED RESULTS - RELATIVE EFFECT OF THE RECOMMENDED PLAN ON PEAK CONCENTRATIONS DURING DESIGN STORM EVENTS [1]

| | Percent Reduction of Peak Coliform Concentrations From Existing Situation to Recommended Plan [2] | |
|---|---|---|
| | Carson Beach Segment 23, 4 | Mouth of Neponset River Segment 20, 2 |
| 1-Year, 6-Hour Storm Event | 60 | 70 |
| 3-Month, 4-Hour Storm Event | 98 | 70 |
| 1-Month, 4-Hour Storm Event | 99 | 70 |

Notes:

[1] From Table 3 of Hydroscience's February 29, 1980 time-variable modeling work submittal.

[2] Approximate percentages, based on values shown in Table XII-9.

Figures XII-31, 32 and 33 show the calculated total coliform concentrations as a function of time for the two monitored segments during the design storm events. It has been assumed that the storm event begins at 18.6 hours, which is slack before ebb tide (i.e., high tide). The HMC model was run for 1.65 tidal cycles (18.6 hours) using all continuous loading inputs to spin up to the background conditions. For Carson Beach during the 3-month, 4-hour, and 1-month, 4-hour storm events, the calculated total coliform concentrations are below the allowable limit throughout the storm's duration. For the 1-year, 6-hour storm, which exceeds the storage/containment facility's design capacity, the limits are exceeded for approximately 28 hours.

The recommended plan results in a decrease in the violation period time in the Mouth of the Neponset River segment as tabulated below:

| Design Storm | Approximate Violation Period, hours | |
|---|---|---|
| | Existing Situation | Recommended Plan |
| 1-year, 6-hour | 43 | 32 |
| 3-month, 4-hour | 40 | 25 |
| 1-month, 4-hour | 31 | 25 |

Based on the above HMC's calculated results, the SB water quality total coliform standards will be exceeded in the modeled segment for the three design storms.

Short-Term Assessment - Southern Section of Dorchester Bay. The HMC time-variable modeling efforts clearly show the improvements to the water quality in the northern section (i.e., South Boston beach areas) of Dorchester Bay if the recommended plan is implemented. However, because of the complexity of the water column movements in the southern section of the bay, which can also be defined as the Neponset River estuary, and the nature of the recommended plan in the Dorchester area, additional evaluations were performed to supplement the HMC results. Investigations of the expected extent of dispersion, advection and dilution of the CSO's with respect to resulting total coliform concentrations were performed by the Dorchester Bay Area Consultant.

Shown in Figure XII-34 are the locations of the water body segments used for analysis. Referring to this figure, the following facts described the existing situtation.

1. Estuarine flows are usually greater, by an order of magnitude, than the CSO discharges. Accordingly, varying CSO inputs would have a minimum impact on the tidal flow pattern.

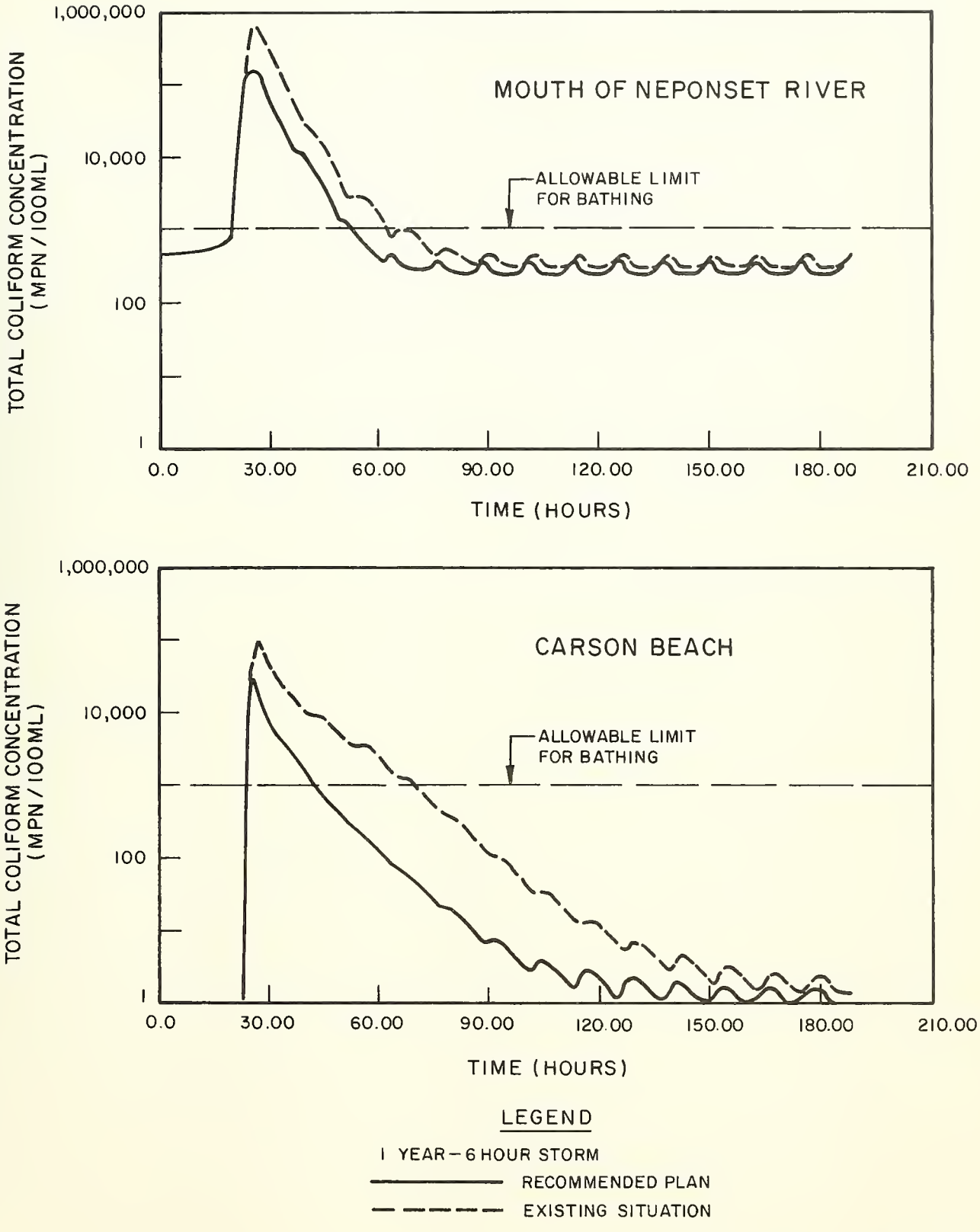


FIG. XII-31 TIME VARIABLE WATER QUALITY RESULTS– 1-YEAR, 6-HOUR STORM EVENT

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METROPOLITAN DISTRICT COMMISSION
COMBINED SEWER OVERFLOW PROJECT
DORCHESTER BAY AREA FACILITIES PLAN

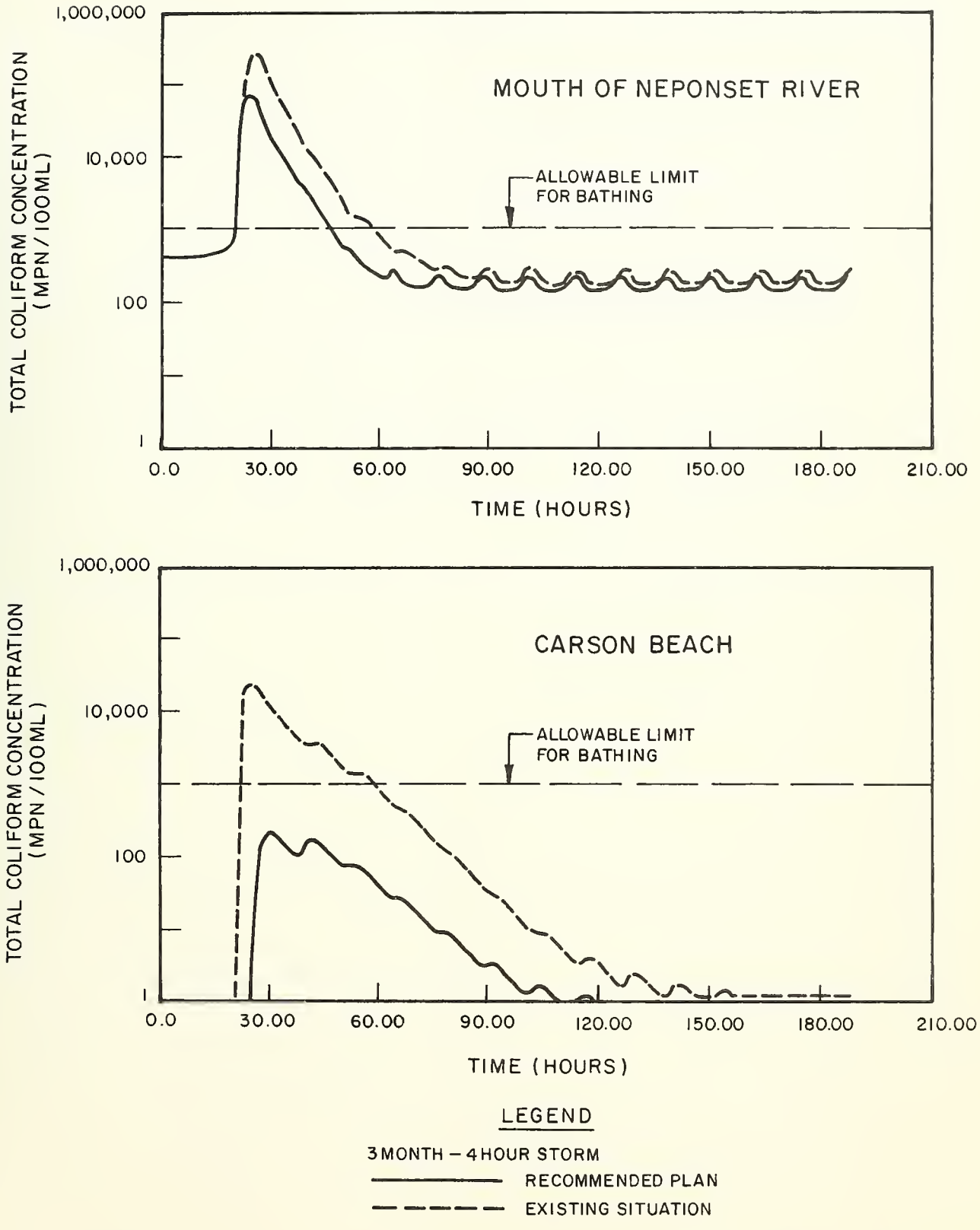


FIG. XII-32 TIME VARIABLE WATER QUALITY RESULTS– 3-MONTH, 4-HOUR STORM EVENT

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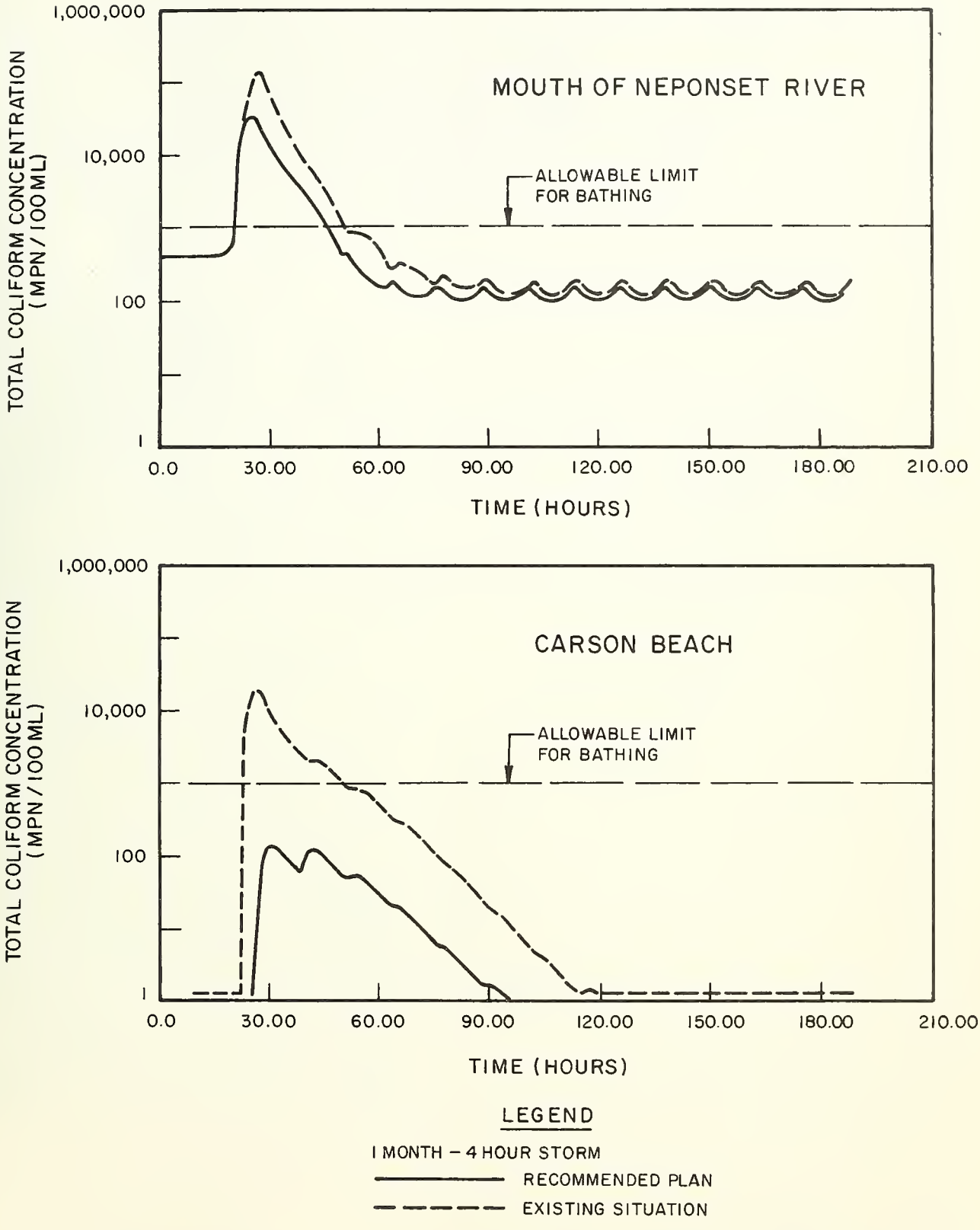


FIG. XII-33 TIME VARIABLE WATER QUALITY RESULTS– 1-MONTH, 4-HOUR STORM EVENT

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DORCHESTER BAY AREA FACILITIES PLAN

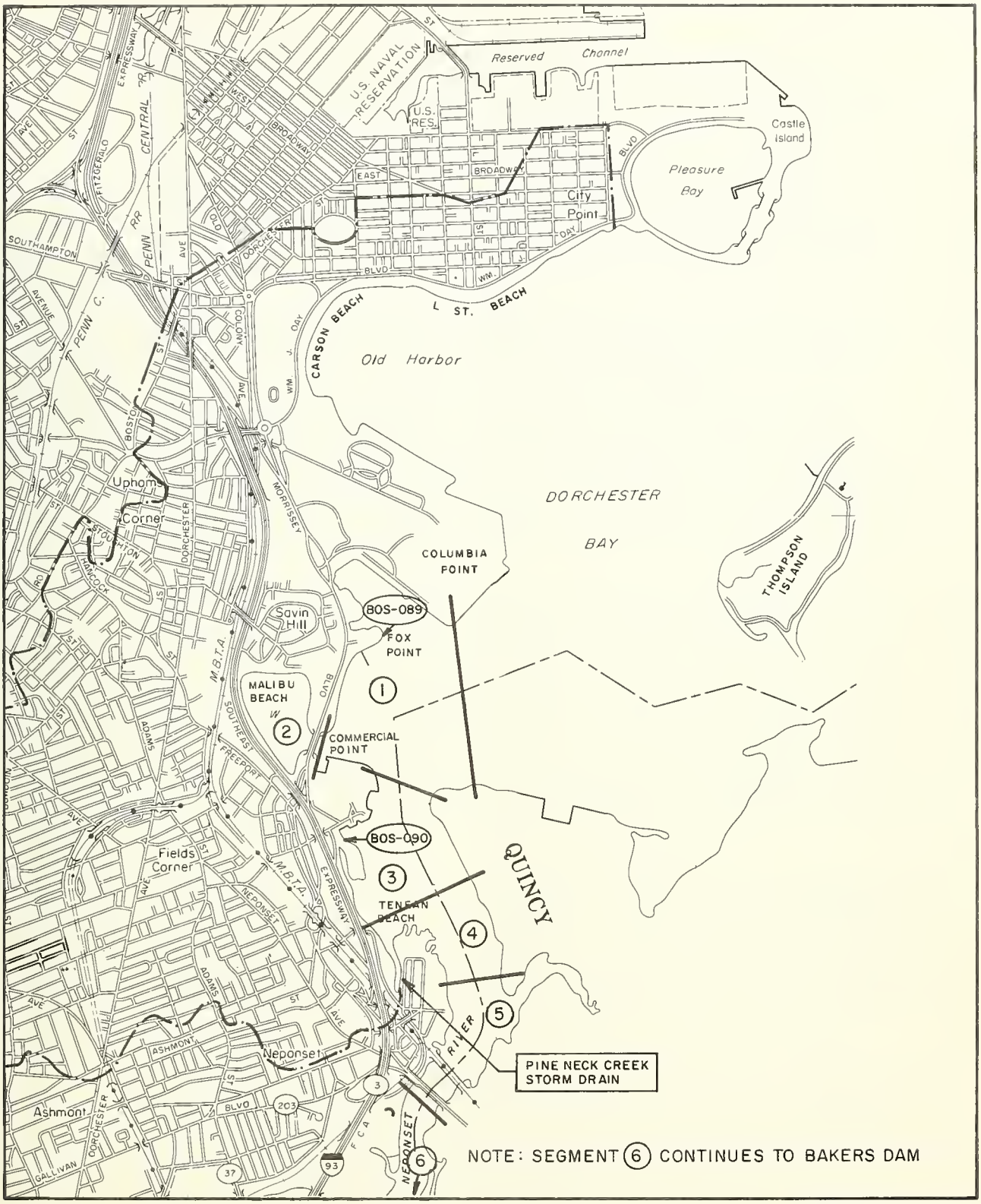


FIG. XII-34 WATER BODY SEGMENTS - SOUTHERN SECTION OF DORCHESTER BAY

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METROPOLITAN DISTRICT COMMISSION
COMBINED SEWER OVERFLOW PROJECT
DORCHESTER BAY AREA FACILITIES PLAN

2. The large area of intertidal marsh in Segment 6 (Baker Dam to Neponset Circle) is an important feature, because it ensures strong, persistent upstream and downstream flows on each tide for the CSO's and Neponset River discharges.

3. Currents can sweep effluent from Segment 1 to Segment 6 in 3 to 4 hours, and back again in another 3 to 4 hours. At a coliform decay rate of 3 per day, a reduction in bacterial numbers in the order of 40 to 64 percent is possible in 4 to 8 hours, respectively.

4. The existing CSO outlets are open-ended conduits, located at about midtide level and a short distance offshore, with no multiport diffusion. Initital dilution rates range between one and ten.

5. CSO related contamination of Segment 2 (Malibu Beach Basin) is mostly from BOS-089 (Fox Point) on the flood tide, although effluent from BOS-090 (Commercial Point) after ebbing to Segment 1 from Segment 3 may contribute to the impact upon flooding to the basin. About 20 percent of Segment 1's volume enters the Malibu Beach Basin on the flood tide.

6. The chances are greater than 50 percent that the storm will start at a tide stage such that BOS-089 effluent collecting in the Malibu Beach Basin will be diluted by a factor of less than 1000. If the total coliform concentration of the CSO discharge is $5 \times 10^6$ per 100 ml. (i.e., worst case situation), concentrations in the basin will likely exceed 5,000 per 100 ml at high tide, for rainfalls starting between 0 and 6 hours before the time of low tide. The greatest coliform concentrations at high tide in the basin occur when the storm starts 1.5 hours before the time of low tide. When storms begin during the flood tide, BOS-089 effluents do not collect in the basin to any great extent.

7. Outlet BOS-090 at Commercial Point is about 2,000 feet downstream of Tenean Beach. For a 1-month, 4-hour storm (i.e., maximum storm event during a typical bathing month) starting at any time between 6.5 hours before low tide to 2.5 hours after low tide, dilutions at Tenean Beach will range from 50 to 1,000. If the CSO discharge coliform concentrations are $5 \times 10^6$ per 100 ml, the resulting concentrations at Tenean Beach will range between $10^4$ to $10^5$.

8. Fox Point (BOS-089) and Commercial Point (BOS-090) CSO discharges occurring from 0 to 6 hours after the time of low tide will, after advection upstream to Section 6, be advected downstream again to be discharged to Dorchester Bay between 7 and 12 hours after the time of low tide. Discharges occurring 7 to 11 hours after low tide will enter Dorchester Bay after a lag of about 1 hour. Therefore, regardless of the start time and duration of an overflow event with respect to low tide, diluted CSO effluent will be discharged into Dorchester Bay during 4 to 6 hours of ebb. The age of the CSO effluent entering the bay will range from 0 to 12 hours.

9. Total coliform "disappearance" factors resulting from die-off and settling range from 0.20 to 0.88 for a decay rate of 3 per day and travel times of 1 to 13 hours, respectively.

10. If the Fox Point and Commercial Point CSO volumes for the 1-month, 4-hour design storm were completely mixed with the low tide volume, the dilution ratio in the estuary area would be about 1:500.

The recommended plan's three major items in the southern section of Dorchester Bay are: (1) the Fox Point screening/disinfection facility: (2) the Commercial Point screening/disinfection facility; and (3) the relocation of the existing Pine Neck Creek separate storm drainage system's outlet. As discussed previously, Items (1) and (2) would screen CSO effluent for all storms up to the 1-year, 6-hour event, and also disinfect the effluent during bathing seasons. Thus, by reducing the total coliform concentration of the CSO effluent, the available dilution ratios would result in a water column probably having acceptable total coliform concentrations. Because of the numerous variables (i.e., wind direction, tidal stage and the corresponding volume of the bay, size of the storm events, etc.) involved which impact on the bay's resulting water quality, it is difficult to estimate the precise total coliform levels at the bathing beaches immediately after an overflow event. Accordingly, the probable, or those which are most likely to occur , coliform concentrations have been estimated. Item (3) takes advantage of the available advection directions and dilution volumes of the Neponset River.

Shown on Figure XII-35 is a plot comparing the estimated BOS-089 outlet's 1-month, 4-hour storm event CSO volume to that of the Malibu Bay Basin under varying tidal conditions. The minimum dilution ratio calculated is about 600, which occurs 1.5 hours before the time of low tide. Under the recommended plan, the BOS-089 CSO effluent would be disinfected during the bathing season. Assuming the CSO's total coliform concentration is $2 \times 10^6$ MPN per 100 ml (actual field tested concentration), and a disinfection kill efficiency of 75 percent, the resulting concentration in the Malibu Bay Basin at the time of minimum dilution would be about 850 MPN per 100 ml. This estimate is considered realistic, although it neglects coliform die-off during travel time, which averages about 1.5 hours over the 2,300 foot distance, and the fact that the CSO effluent will not be uniformly mixed with the basin's water. The chlorine residual in the basin is estimated to be less than the recommended 0.01 water column limit, assuming that the end-of-pipe chlorine residual will be 2 mg/l.

Tenean Beach is located about 2,000 feet upstream of the BOS-090 outlet. Dilution ratios for CSO volumes generated by a 1-month, 4-hour storm from the outlet starting any time between 6.5 hours before to 2.5 hours after low tide to the Tenean area's water body, ranges from 50 to 1000, depending upon the storms starting time and the tidal cycle. Using an average dilution ratio of 475, and assuming the CSO total coliform concentration is $2 \times 10^6$ MPN per 100 ml and a 75% disinfection kill efficiency, the resulting total coliform concentration in the Tenean Beach area would be about 1,000 MPN per 100 ml. During the lower dilution periods, or during low tide,

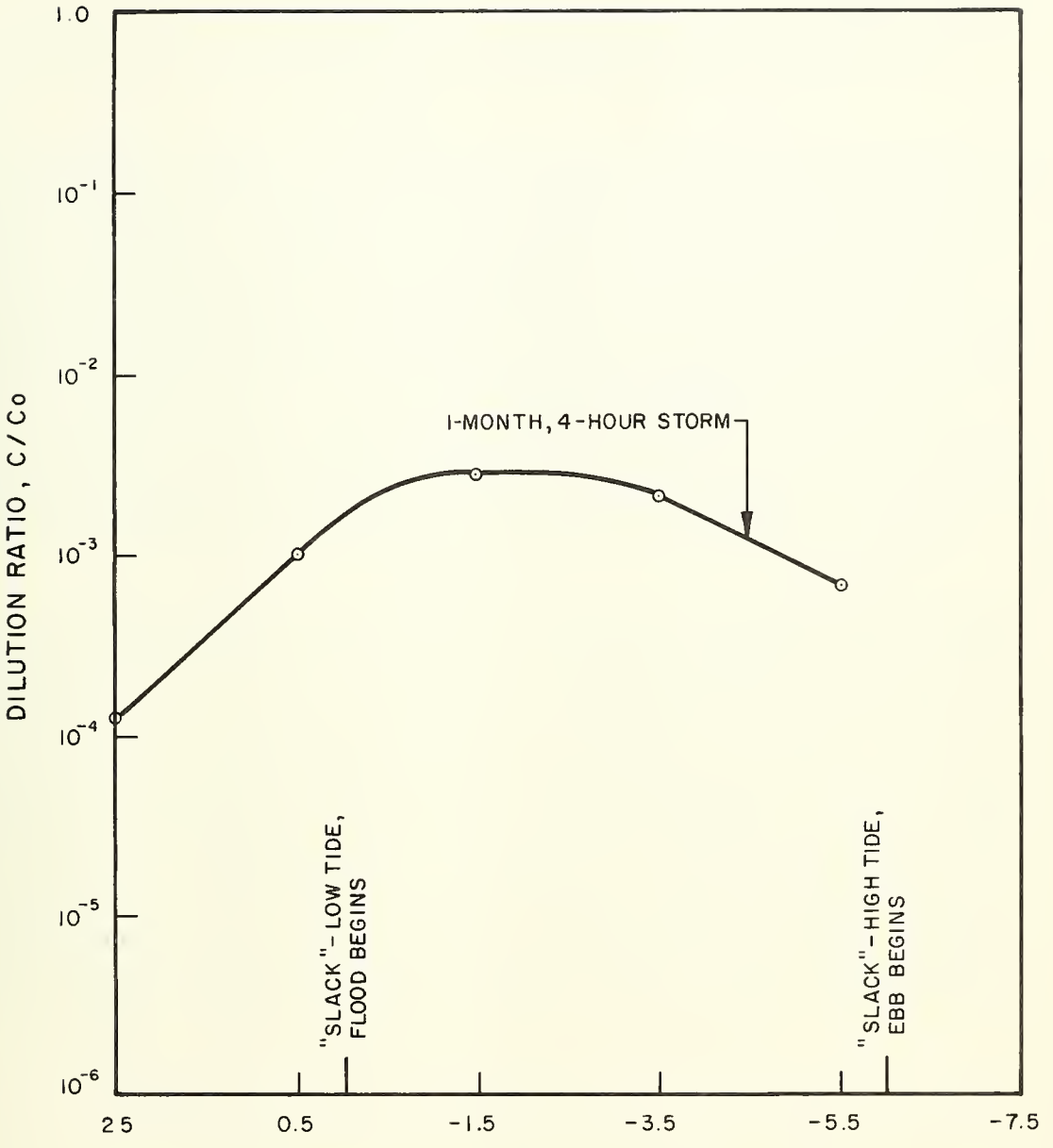


FIG. XII-35 DILUTION RATIOS OF MALIBU BAY BASIN AND BOS-089 CSO OUTLET FOR 1-MONTH, 4-HOUR STORM EVENT

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METROPOLITAN DISTRICT COMMISSION
COMBINED SEWER OVERFLOW PROJECT
DORCHESTER BAY AREA FACILITIES PLAN

total coliform concentrations would exceed the bathing limit of 1000 MPN per 100 ml. However, during low tide, the beach's available swimming area is negligible, with essentially only the Neponset estuary's navigation channel flowing. If BOS-090's end-of-pipe chlorine residuals can be maintained at 2 mg/l, the area's water column chlorine residual will range from 0.04 mg/l to 0.002 mg/l, depending upon the time of the storm's start and the tidal cycle.

As discussed previously, the long-term 50th percentile total colfiorm concentration is calculated to be less than the allowable 700 MPN per 100 ml for shellfishing. However, during and for a minimum period after an overflow event in the southern section of Dorchester Bay, the current shellfish harvesting area's water column will have total coliform concentrations greater than the allowable limit. Also, in the immediate vicinity of the CSO outlets during the summer months, the chlorine residual concentrations will exceed the recommended allowable limit of 0.01 mg/l. On a long-term basis, such a situation is expected to occur 6 times per summer season, between June 1 and September 1. Assuming the statistical long-term typical storm during this period is the 3-month, 4-hour event, the screening/disinfection facilities are expected to be operated for about a 6-hour period. Chapter X of Volume II discusses further the environmental consequences of the disinfection process.

The existing Pine Neck Creek separate storm drainage system discharges directly into the Tenean Beach waters. Under the recommended plan, the discharge point would be relocated to the Neponset River. In order for this storm water discharge to impact on Tenean Beach under the recommended plan, it must flow approximately 2,000 feet downstream on the ebb tide, then flow onto Tenean Beach on the ensuing flood tide. If this one pollutant sidestream could be isolated, enough dilution is available 50 percent of the time to result in total coliform concentrations at Tenean Beach less than 100 MPN per 100 ml. However, as noted previously, the Pine Neck Creek storm drainage system's total coliform load is minimum compared to those of the Neponset River during a storm event. Although the recommended plan takes advantage of the advection and subsequent dilution volumes within the southern section of the bay to mitigate the impact of Pine Neck Creek storm drainage on Tenean Beach, the Neponset River pollutant loads dictate the area's water quality.

## Summary of Estimated Costs of Recommended Plan

A summary of the estimated capital and operation and maintenance costs of the recommended plan is shown on Table XII-11. A discussion on financing, including those portions of the costs expected to be borne by federal and state grant allocations, is presented in Chapter XIII. Also presented in Chapter XIII is a recommendation on staging of items included in the recommended plan.

TABLE XII-11 SUMMARY OF ESTIMATED CAPITAL AND ANNUAL OPERATION AND MAINTENANCE COSTS OF THE RECOMMENDED PLAN

| Item | Estimated Capital Cost[1] | Est. Annual O&M Costs[2] |
|---|---|---|
| I. Structural Recommendations | | |
| South Boston | | |
| Consolidation Conduit | $ 9,620,000 | $ - |
| Storage/Containment Facility | 18,380,000 | 38,000 |
| Dorchester | | |
| Hoyt St. Regulator Modification | 90,000 | - |
| Commercial Point Screening/Disinfection Facility | 3,030,000 | 22,000 |
| Fox Point Screening/Disinfection Facility | 2,740,000 | 21,000 |
| Pine Neck Creek Storm Drain Relocation | 2,340,000 | - |
| Subtotal | $36,200,000 | $ 81,000 |
| II. Non-Structural Recommendations | | |
| Dry Weather Overflow Abatement Program | $ 100,000 | |
| Best Management Practices | 25,000 | 200,000 |
| Subtotal | $ 125,000 | $ 200,000 |
| III. Additional Recommendations | | |
| Cleaning of CSO Conduits | $ 300,000 | $ - |
| Dredging | 40,000 | - |
| Landfill at CSO Outlet BOS-090 | 20,000 | |
| Subtotal | $ 360,000 | $ 0 |
| TOTAL | $36,685,000 | $ 281,000 |

NOTES:

1 Based on June 1979 price levels (ENR 2900) and includes an allowance for engineering and contingencies.

2 Based on 1979 price levels.

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